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MODEL STUDIES OF RIVER CONTROL METHODS

V.M. APPELT

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MODEL STUDIES OF RIVER CONTROL METHODS

A DISSERTATION
SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF MASTER OF SCIENCE

FACULTY OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

BY

V. M. APPELT

UNDER THE DIRECTION OF
T. BLENCH

EDMONTON, ALBERTA.

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MODEL STUDIES OF RIVER CONTROL METHODS

Abstract

Model studies on the University of Alberta river tray confirmed that small-scale rivers of different types can be produced by suitable control of discharge and sediment load. Such models may be used to demonstrate the effects of engineering works on the behaviour of full-sized rivers of the same types.

The experiments described herein demonstrated a means of protecting bridge abutments from erosive attack, the action of artificial islands as an aid to river rectification, and the effects of short spurs as bank protection works.

Concurrently, a glass-walled flume was used to develop flume study techniques and to initiate studies of the properties and actions of sand transported as bed-sediment.

Some of the difficulties encountered indicated the failings of present equipment and suggested how it might be improved for future studies along the same lines.

PART I - MODEL RIVER STUDIES

PREAMBLE

The importance of model studies in river control is coming to be recognized to an ever greater degree. These studies provide a means of producing cheaply and quickly the answers to such questions as, "What will be the effects of a certain course of action?" or, "Which of several courses will produce most effectively and economically the desired results?"

Many of the problems confronting the river engineer require only a qualitative answer. In many cases, it is then not necessary to build a formal model of the river reach being studied if the conditions of flow and sediment movement can be reproduced by other means. Such means are provided by a river tray such as the one constructed in 1953 at the University of Alberta.

It is necessary first of all to regulate the flow of water and sediment so as to produce conditions typical of the river under study. Success in this will ensure that at some time a part of the model will reproduce the particular site conditions under which the engineering work is to be done. It is then required only to place on the model a suitably scaled miniature of the work and to observe and interpret the results.

The limitations of this type of model study must be understood if usable answers are to be obtained. One of these limitations is created by the fact that measurements of different quantities "scale down" differently. The discharge scale, for example, is not the same as the width, or time scale.

Thus, as has been noted by most makers of river models, the depth must be exaggerated in the model. The effect is to steepen the slope and to require banks to be nearly vertical. The steepness of the banks is however limited by the natural angle of repose of the bank material, so that an unavoidable distortion may result.

This may also affect quantitative measurements such as the relative depth of scour holes. The sides of the holes cannot stand as steeply as is required by the vertical exaggeration. In spite of this, even fairly small models give scoured depths which are in reasonable accord with those found in nature.

Regime theory affords the best means available of calculating model scales. An excellent discussion of this use of the regime equations is given in Ref. 1. This approach has been used in the present experiments to interpret the quantitative measurements made. The bibliography lists other references to the use of models in the solution of particular problems. (See Refs 2 and 3).

DESCRIPTION OF EQUIPMENT

The river tray is described in detail in Ref. 4. It is essentially a shallow tank, 20 x 49 feet, divided longitudinally by a diaphragm. Each of the two halves thus formed has its own recirculating pump, control valve and venturi meter. Thus two entirely separate miniature rivers can be formed in whatever bed material is placed in the trays.

At present two grades of sand are used. That in the east half has median grain size of approximately 0.27 millimeters, and that in the west half 0.41 millimeters. Both sands were taken from the Elk Island Pit, and appear to be more truly beach than river sands.

A photograph, Plate 19, shows the general layout, with the grading gantry in the background. This photograph was taken at the conclusion of the tests described herein, and indicates something of the difference in the way the two rivers developed.

PLANNING AND DESCRIPTION OF EXPERIMENTAL WORK

Previous experiments at this University (Ref. 4) indicated that in order to ensure rapid initial development, the slope set on the model should be considerably in excess of regime slope. Accordingly, both the sand surface in the tray and the initial straight channel were graded to four times the slope calculated from the regime equations for a discharge of 0.2 cubit feet per second and no bed load. The appendix lists the equations necessary for these calculations.

Some difficulties were encountered from the beginning of the experiments. One of these was due to the entrapping of air in the manometers, resulting in erroneous readings. The discharge could not be kept constant, either because of unsteadiness of pump delivery or a gradual increase of loss of head in the control valves.

INITIAL DEVELOPMENT

The east half of the river tray was started first, with a discharge of 0.09 c.f.s. Bed waves were formed within the first few minutes, and at the initiating bend, erosion of the west bank began almost immediately. Within one day it was evident that the familiar meandering type of sand river was developing.

The west side of the tray, containing the coarser sand, was given an estimated initial discharge of more than 0.2 c.f.s., which was the calculated peak flood discharge. The channel at first ran slightly less than bank-full, and meandering started at a much faster rate than in the east half. Nine hours after the flow was started the tray contained a miniature river of the boulder type. The flow was split into several small channels, with the water surface riffled into a series of V-shaped standing waves. The bed material was moving as a sheet, the bars being smooth and the channel bed showing no dunes at all.

The reason for this action was no doubt the heavy discharge imposed. Large quantities of sediment were carried downstream and through the pump, to be deposited at the head of the channel where an "alluvial fan" was formed. The channel cross-section was flattened and the total flow spread over a width much greater than the original designed width.

This illustrates something of the flexibility of a river tray of this type. Experience and an understanding of the principles involved will enable the experimenter to vary the mode of development of his model river to suit his purposes.

As time went on, the stream in coarse sand showed a marked tendency to change course by the opening of cut-offs without the influence of varied discharges. In the cut-offs, the standing waves mentioned earlier were particularly apparent. At no time did this half of the tray show more than a few bed-waves, and these only in the comparatively slack water areas. These are features first of all of "sheet flow", the state between dunes and antidunes and typical of critical flow conditions. The whole model was illustrative of the braided type of river reach, whose bed is made up of or carries very coarse sediment such as gravel.

The fine sand river on the other hand continued as a meandering stream, the formation of the meander bends progressing rather slowly downstream. There was some steepening of the slope as sediment was deposited in the upstream section. There was very little tendency for cut-offs to form except after the imposition of flood conditions.

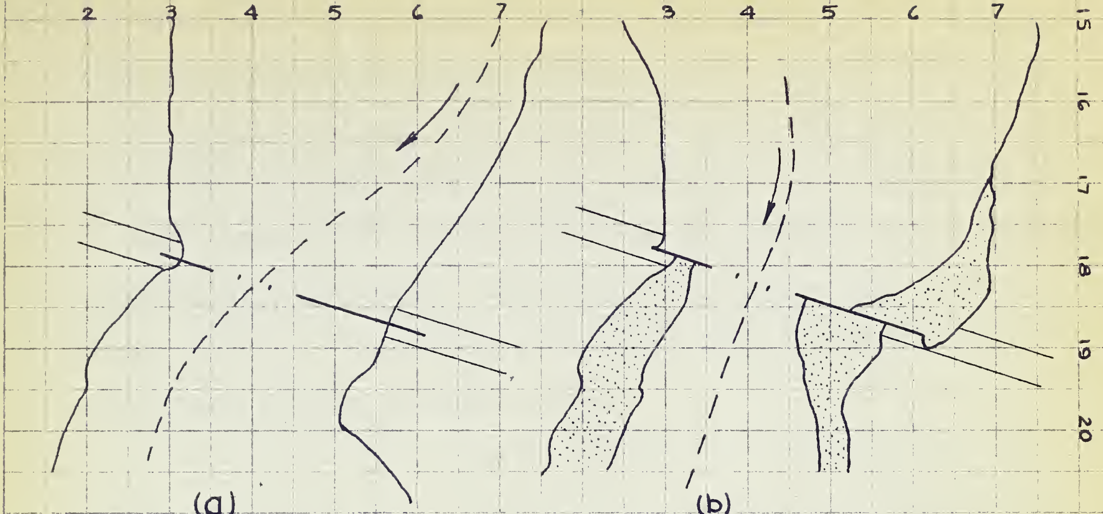
The maximum discharge imposed during the first few months was 0.098 c.f.s., and for the greater part of the time only .02 c.f.s., was flowing.

GUIDE BANKS AS BRIDGE PROTECTION

In order to illustrate the effectiveness of guide banks in protecting bridge abutments against scour, two model bridges were placed on the east half of the tray. The first of these, not equipped with guide banks, was set in place on January 29th in the position shown on Plate 1 (a). The flow conditions here were not unduly adverse, but promised to produce eventual attack on the eastern abutment and roadway.

By February 28th some damage had occurred on this side, but several cycles of flood and low flow had caused a cut-off to form upstream, diverting the main flow to nearer mid-stream. Erosion continued slowly in spite of this lessening of the attack, and in addition, some damage occurred to the roadway on the west side, as shown in Fig.(c). Both abutments now had deep scour holes at their ends and along their upstream sides. On March 14th the eastern roadway was breached. Rapid flow with consequent serious erosion took place. On the following day the other side was breached as well.

In contrast the use of guide banks of length equal to span on the downstream bridge protected the abutments and roadway from attack in spite

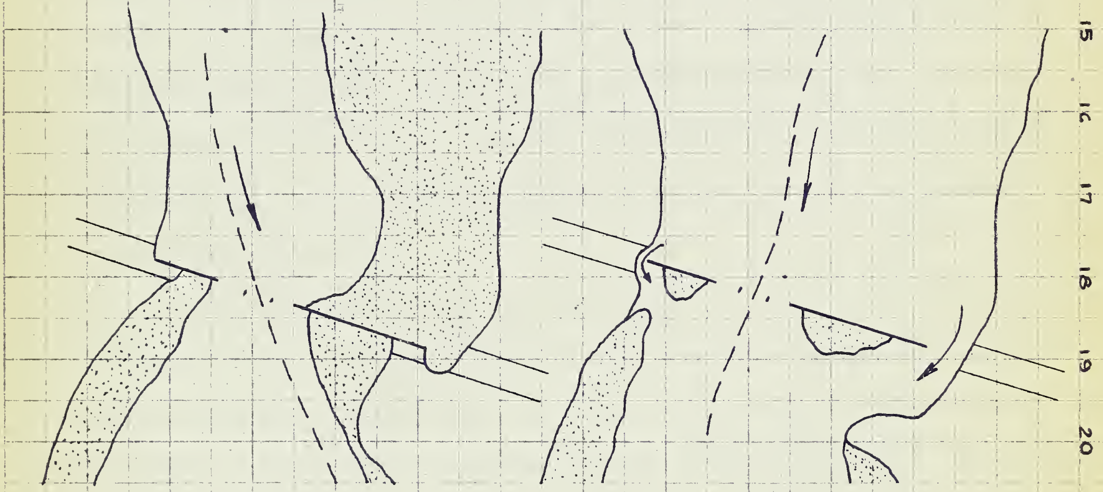


(a)

29 Jan. 1955

(b)

28 Feb. 1955



(c)

5 March, 1955

(d)

15 March 1955

U. OF A. RIVER TRAY EAST

$Q \doteqdot 10\% \; Q_{max.}$

BREACH OF ROADWAY AT UNPROTECTED BRIDGE

Scale - 1 in. = 2'-0"

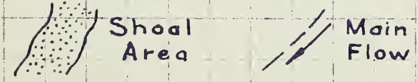


PLATE No. 1

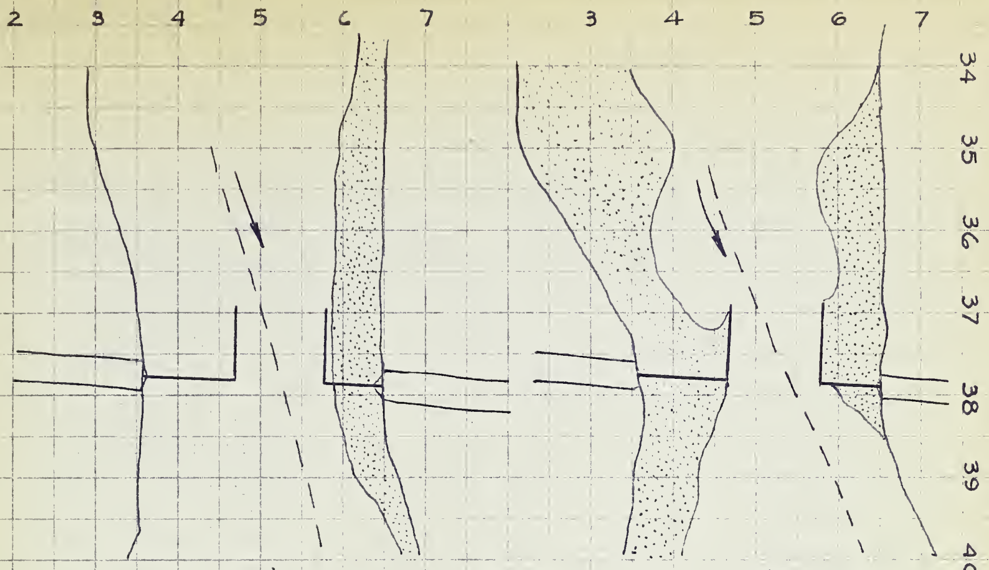
of repeated floods and adverse alignment of flow. Deep scour occurred at the noses of the guide banks and along the sides. Good practice in the full-scale case calls for heavy armouring of the guide bank noses so that it may be said that properly designed guide banks can be completely effective for this purpose. Plate 2 shows the changes which occurred at bridge number two from February 5th to March 15th. The roadway will be seen to have suffered no damage at all.

The action of guide banks is as simple as it is effective. The pocket formed by the guide bank, abutment, and river bank contains water which can do nothing but circulate very slowly. Because of the reduced velocity of flow, there is no bed-movement in the pocket and suspended sediment settles out, resulting, in the case of a river with a heavy suspended load, in the complete "silting-up" of the pocket. Even in the absence of this action, the dead water acts as a buffer in time of flood, so that only slow-moving water is in contact with the bridge roadway and abutments.

Following the breaching of the roadway at Bridge No.1, guide banks of length equal to three-quarters of the span were placed there and the damage repaired. Plate 3 shows the conditions at this bridge on March 15th and again on March 25th, on which date the flow was stopped. Within this 10 day period, two complete cycles of flood were imposed, up to a maximum of 0.2 c.f.s. It will be seen that the guide banks functioned effectively in spite of very adverse alignment of flow for at least half this time.

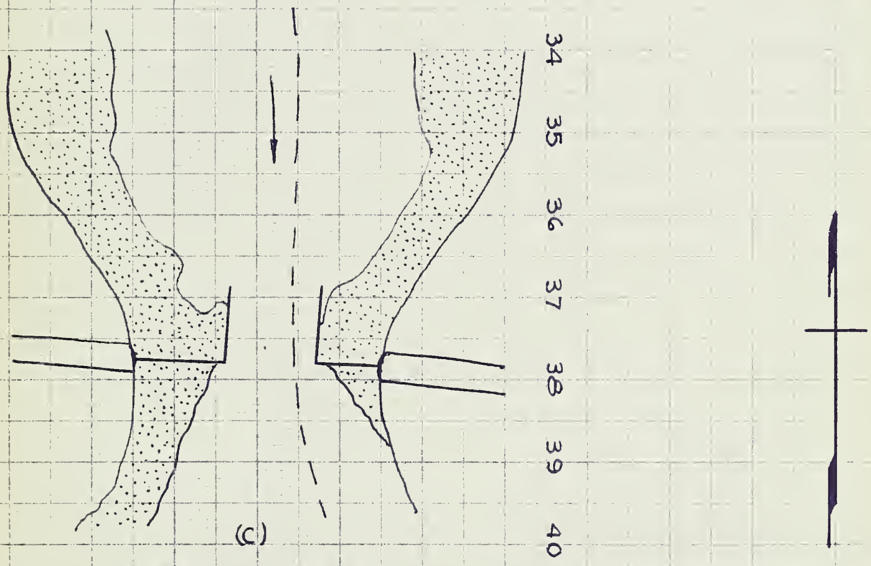
PITCHED ISLANDS AS AN AID TO RIVER CONTROL

Very little literature is available on this subject, most of it being that contained in Ref.5. Even in India, where the idea originated,



(a)
5 Feb.1955

(b)
1 March 1955

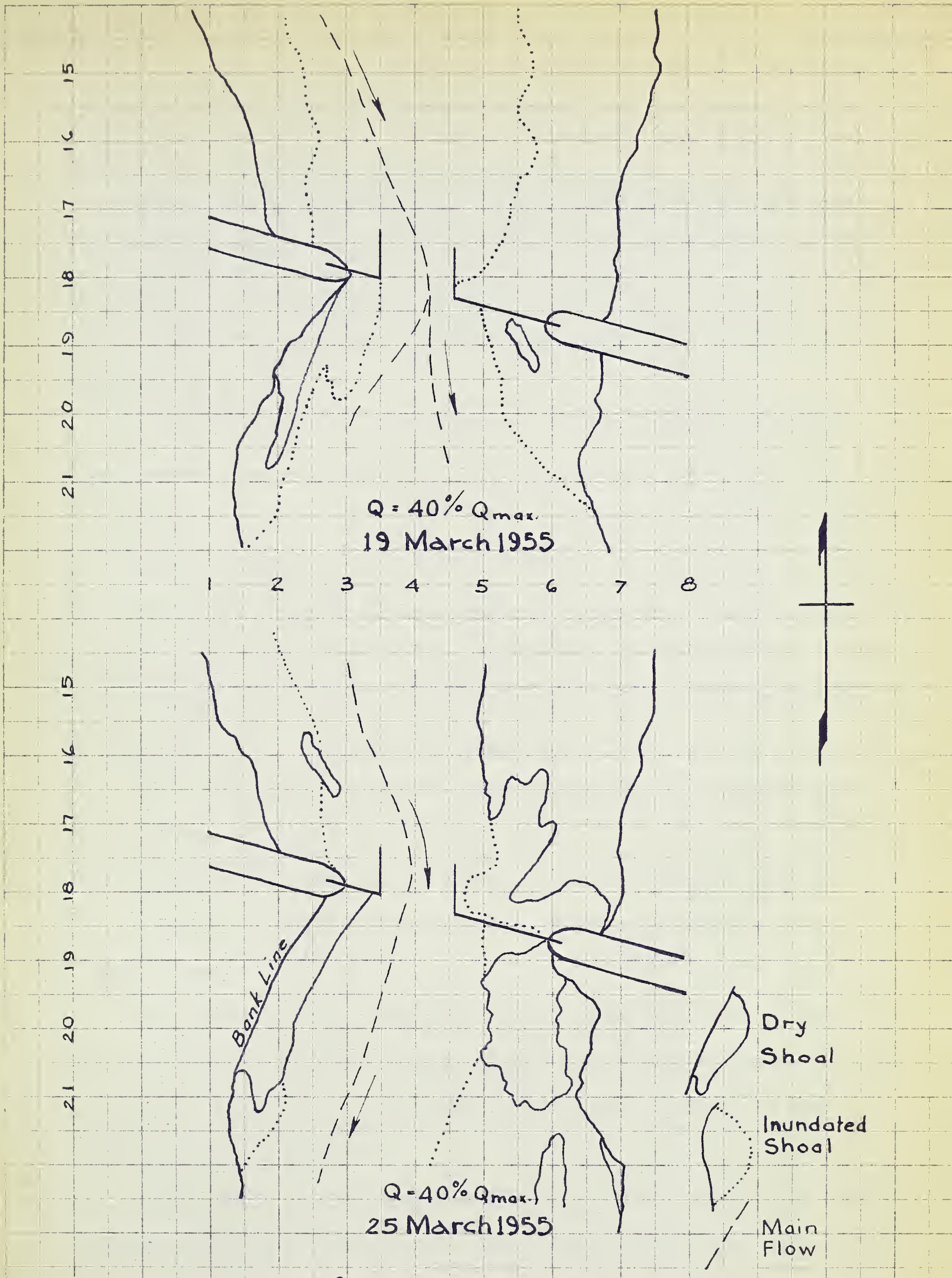


(c)
15 March,1955

U. OF A. RIVER TRAY EAST
Q = 10 % Qmax.
BRIDGE PROTECTED BY GUIDE BANKS
Scale - 1 in = 2'-0"

▨ Shoal Area
↙ Main Flow

PLATE No. 2



Q = 40% Qmax.
19 March 1955

Q = 40% Qmax.
25 March 1955

U. of A. RIVER TRAY EAST
GUIDE BANKS ON UPSTREAM BRIDGE

Scale - 1in. = 2'-0"

PLATE Nº 3

not many attempts appear to have been made to rectify rivers by this means.

One successful utilization of a pitched island is described in the above reference. The situation required the centralization of flow upstream of a barrage, where very adverse conditions had developed. Model studies predicted the success of the attempt and showed the best location and alignment.

A pitched island is essentially an artificial island intended to draw the main flow of current upon itself and away from another part of the river. It is usually constructed in the dry on a shoal, of whatever fill is available, and armoured with heavy stone to withstand attack. Upon the arrival of a freshet, the shoal is inundated and scour takes place alongside the island, thus forming a preferred path for the flow. There may be also a "self-dredging" action set up by eddying flow continuing for some distance downstream of the island.

The action may be likened to that of the head of a T-headed spur. The experience of engineers in India (See Ref.2) has been that the main channel, instead of being deflected by these spurs,tends to stick to their heads. The pitched island may thus function as a T-headed spur without a shank.

The experiments were begun by placing on the east tray on March 1st the three islands shown on Plate 4. These were simply rectangles of wire screen which it was hoped would afford sufficient roughness to create eddies along their lengths.

A full cycle of flood from 0.075 c.f.s., to 0.2 c.f.s., was imposed without any apparent action on the part of the islands. Two more cycles gave, by March 12th, the results shown on Plate 5. The island at station

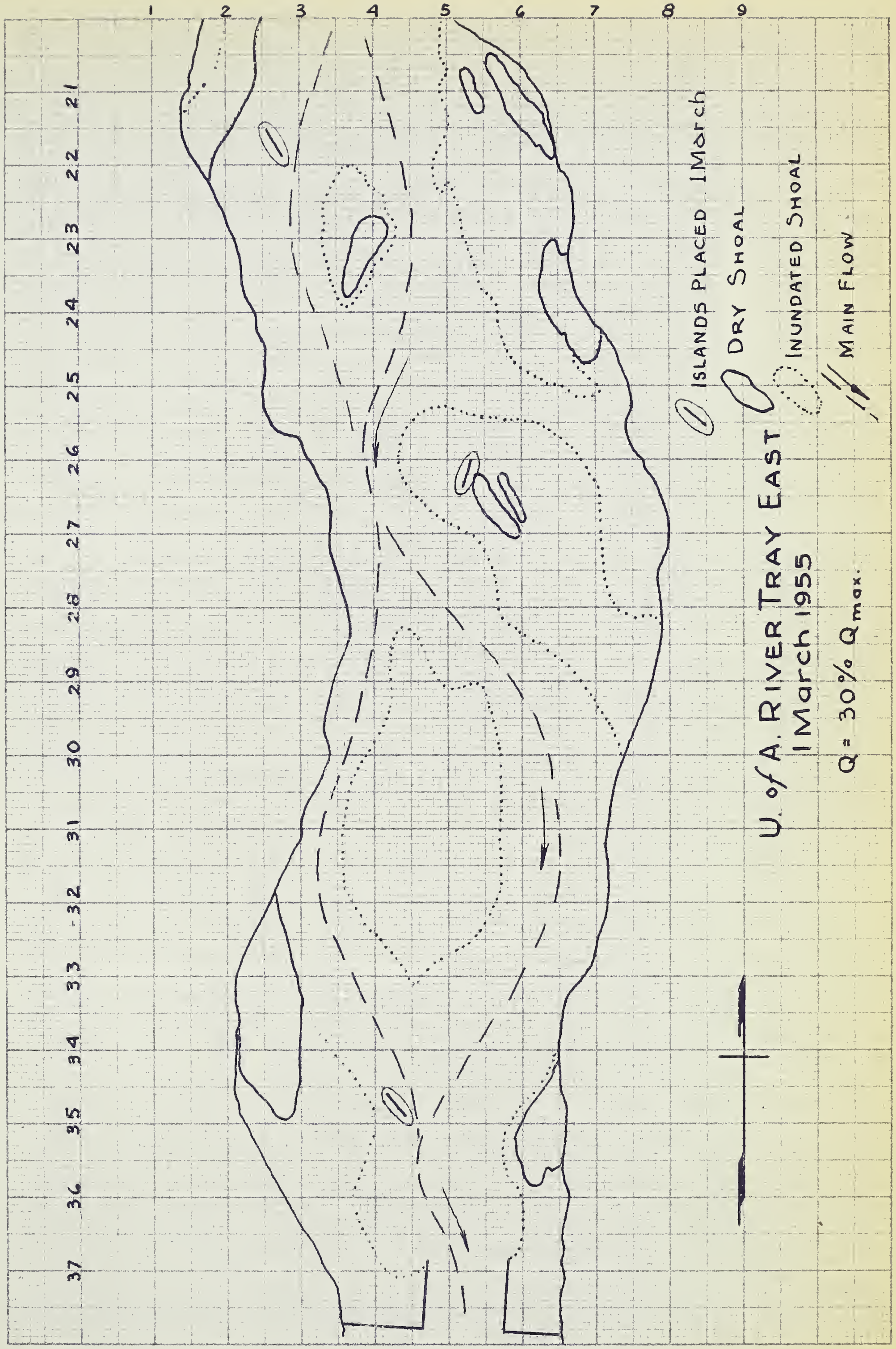

U. of A. River Tray East
1 March 1955
Q = 30% Q_{max}

PLATE No. 4

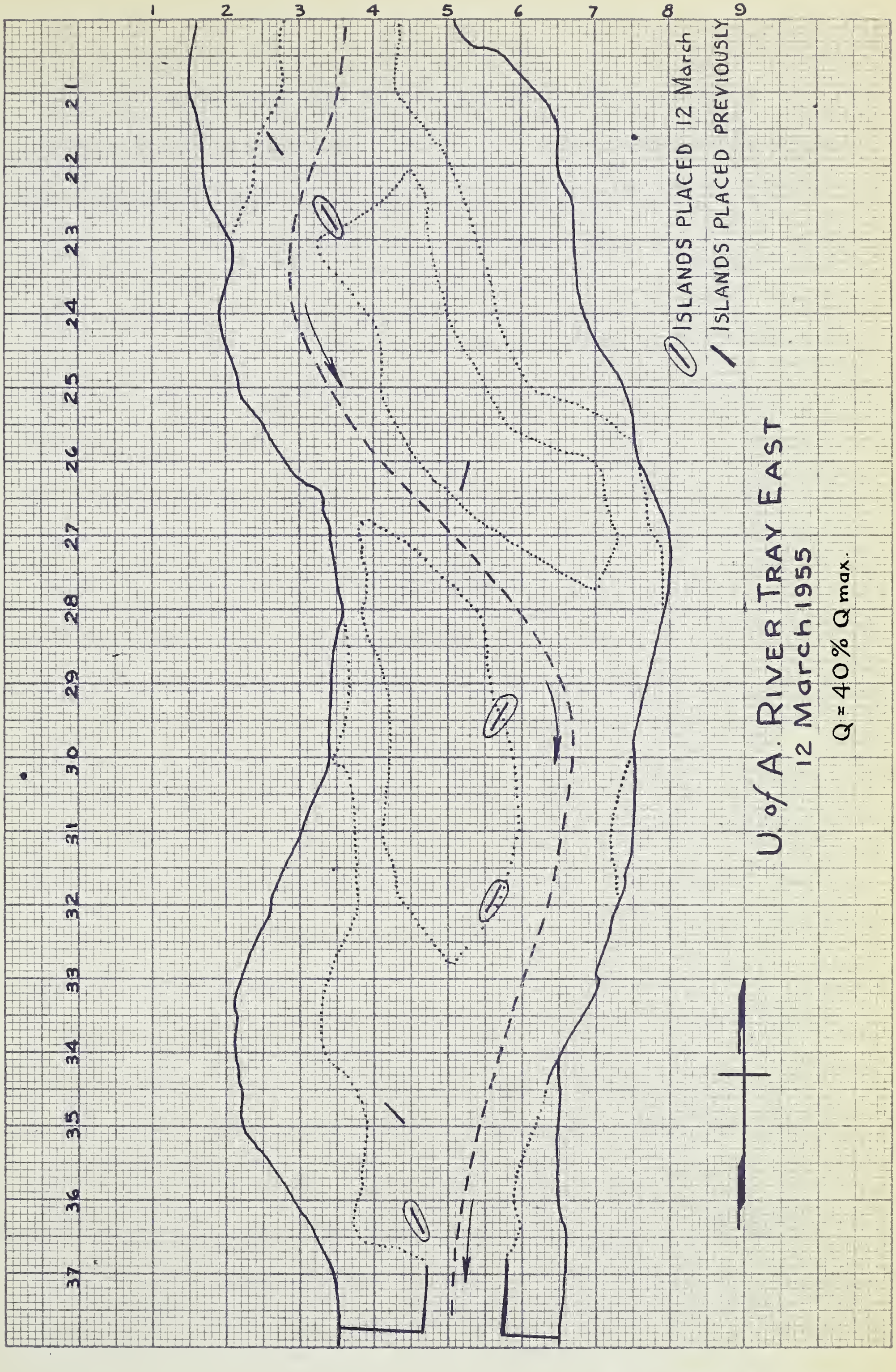

U. of A. RIVER TRAY EAST
12 March 1955
Q = 40% Q max.

PLATE No. 5

26 may have had some influence in drawing the flow upon itself.

Four more islands were placed on March 12th and two floods imposed between that date and March 19th, the resultant changes being as indicated by Plate 6. It is difficult to say how effective the action of the islands was, as some changes in the river would have occurred in any case.

The choice of site for these islands was dictated by the positions of the dry bars, as, in a practical case, access to the site would be required for construction.

The islands placed on March 19th were aligned as shown in an attempt to simplify the arrangement so that the results could be more easily assessed. They consisted of rectangles of 3/8" plywood. Two flood cycles between this date and March 25th resulted in a considerable alteration of the flow path. Plate 7 shows the east side of the channel to have silted heavily, so that the main flow was fairly straight and almost entirely to the west of the line of islands.

Considerable work is necessary to develop the correct techniques for placing these pitched islands most effectively. The geometry and surfact texture of the islands may be found also to affect considerably the action which they produce.

In spite of the uncertainty of the results obtained here, it may be said that pitched islands are worthy of a much fuller investigation. They promise to provide a means of river rectification much less costly than spurs, since construction of the spur shanks is eliminated.

MODEL STUDY OF SHORT SPURS USED AS BANK PROTECTION IN AN ACTUAL CASE

Plates 8 and 9 show the results of an investigation of the use of short spurs as bank protection, modelled after an actual case on a

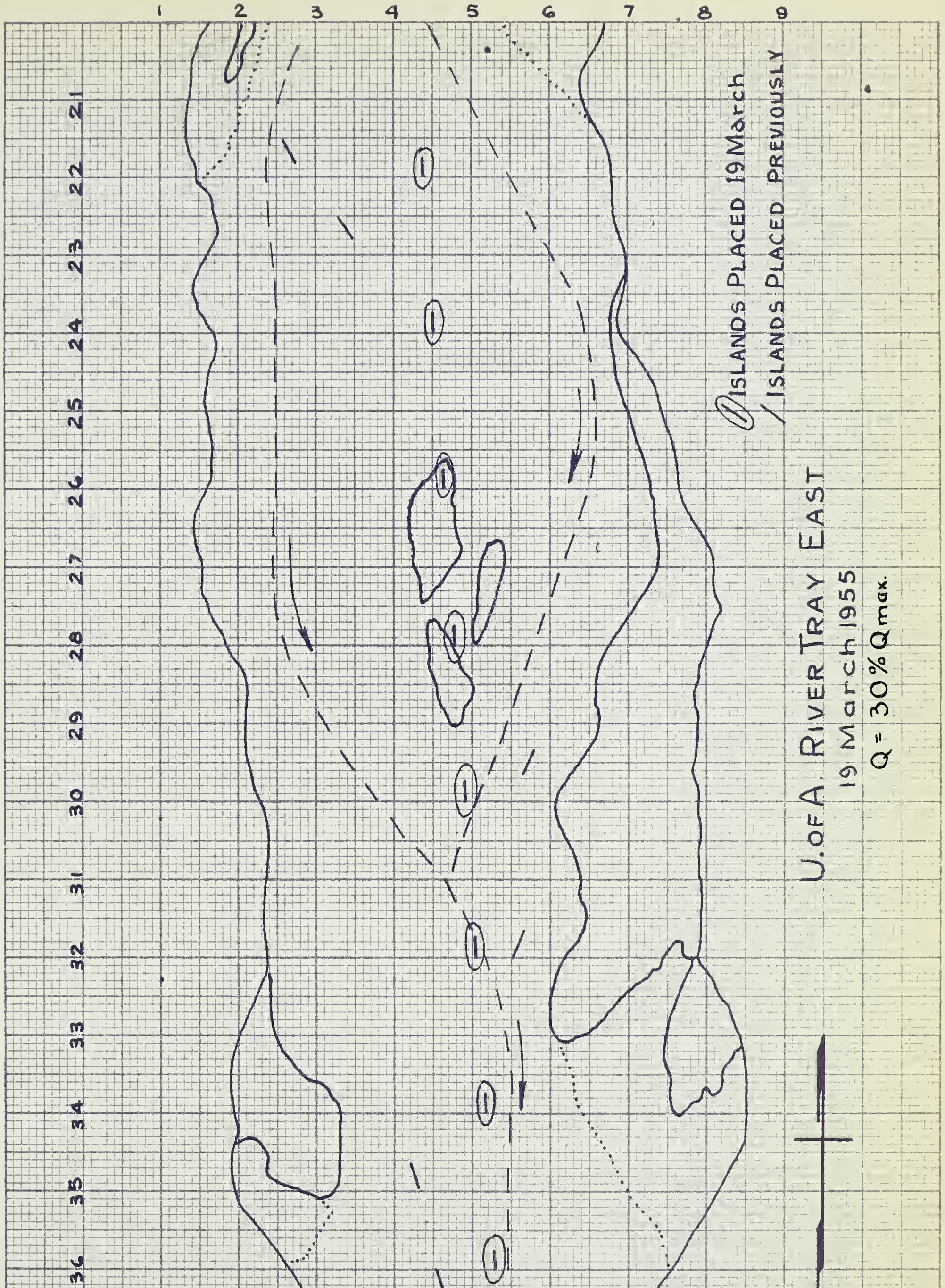
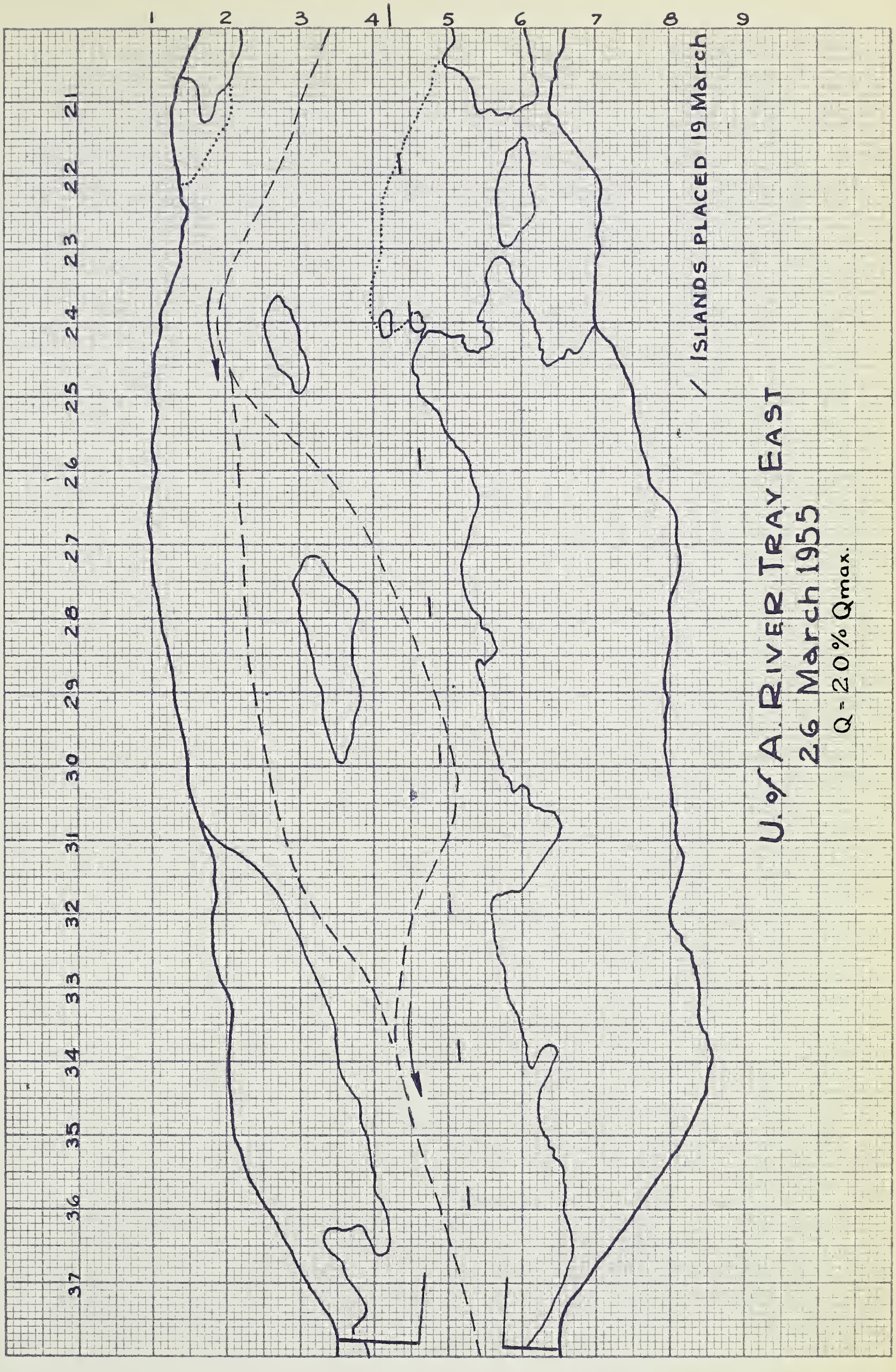
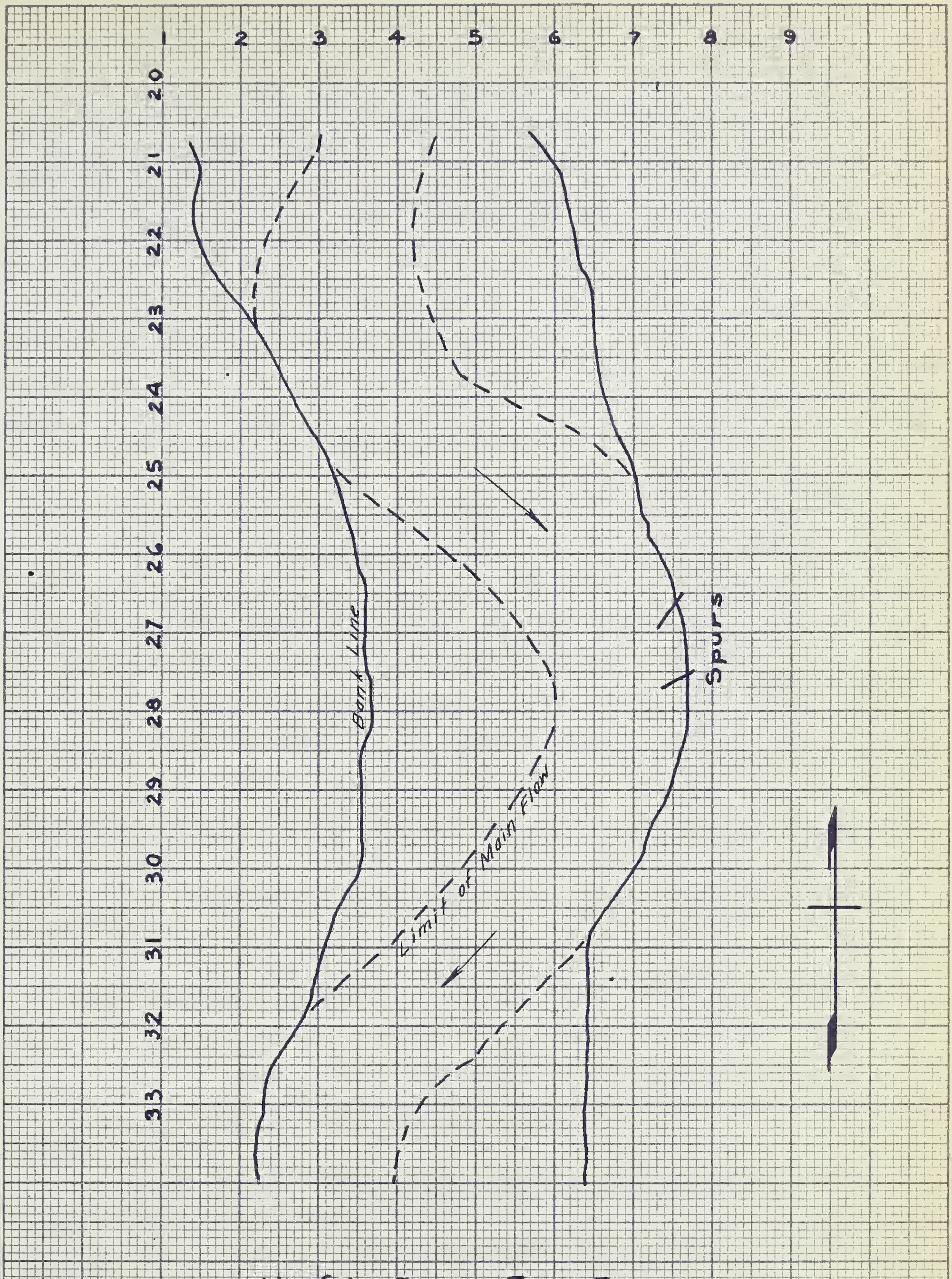

U. of A. River Tray East
19 March 1955
Q = 30% Q max.
ISLANDS PLACED 19 March
ISLANDS PLACED PREVIOUSLY
1
2
3
4
5
6
7
8
9
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37

PLATE No. 6



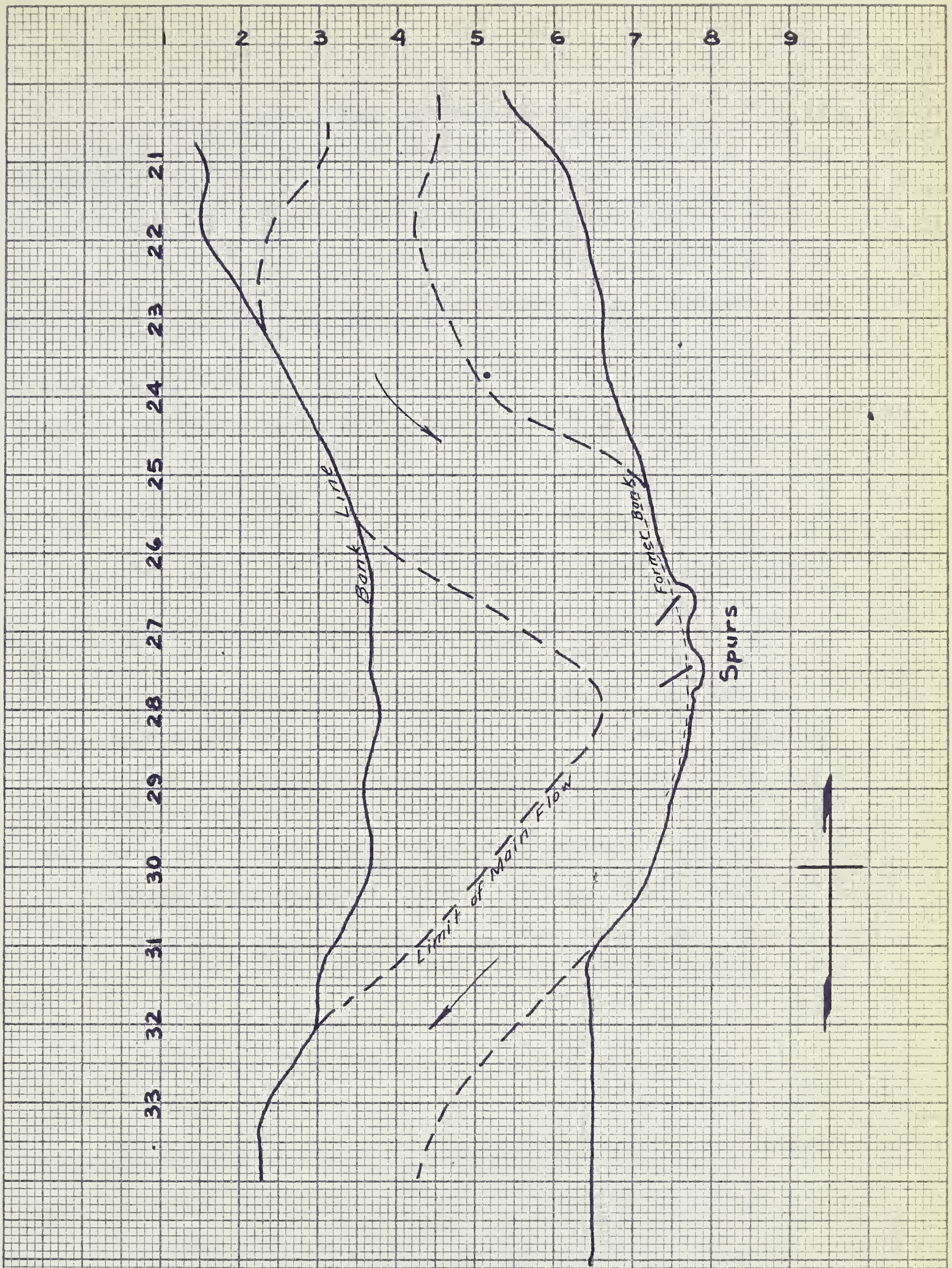
U. of A. RIVER TRAY EAST
26 March 1955
Q = 20% Qmax.

PLATE No. 7



U. of A. River Tray East
Short Spurs Placed 1 Feb. 1955
Scale - 1 in. = 2'-0"

PLATE No. 8



U. of A. River Tray East
Erosion at Spurs 8 Feb. 1955
Scale - 1in. = 2'-0"

PLATE N° 9

northern Alberta river.

Serious erosion during annual spring freshets had resulted in a considerable loss of property on the outside of a meander bend. As a protection measure the two spurs shown were constructed, some years apart. Immediately downstream of the spurs, or "wing dams", erosion was stopped, but upstream the bank continued to suffer attack. Scour at the ends and alongside the structures themselves caused them to settle and partially collapse. Eventually the attack at the shore end forced the provision of further works to protect the spurs themselves as well as the bank.

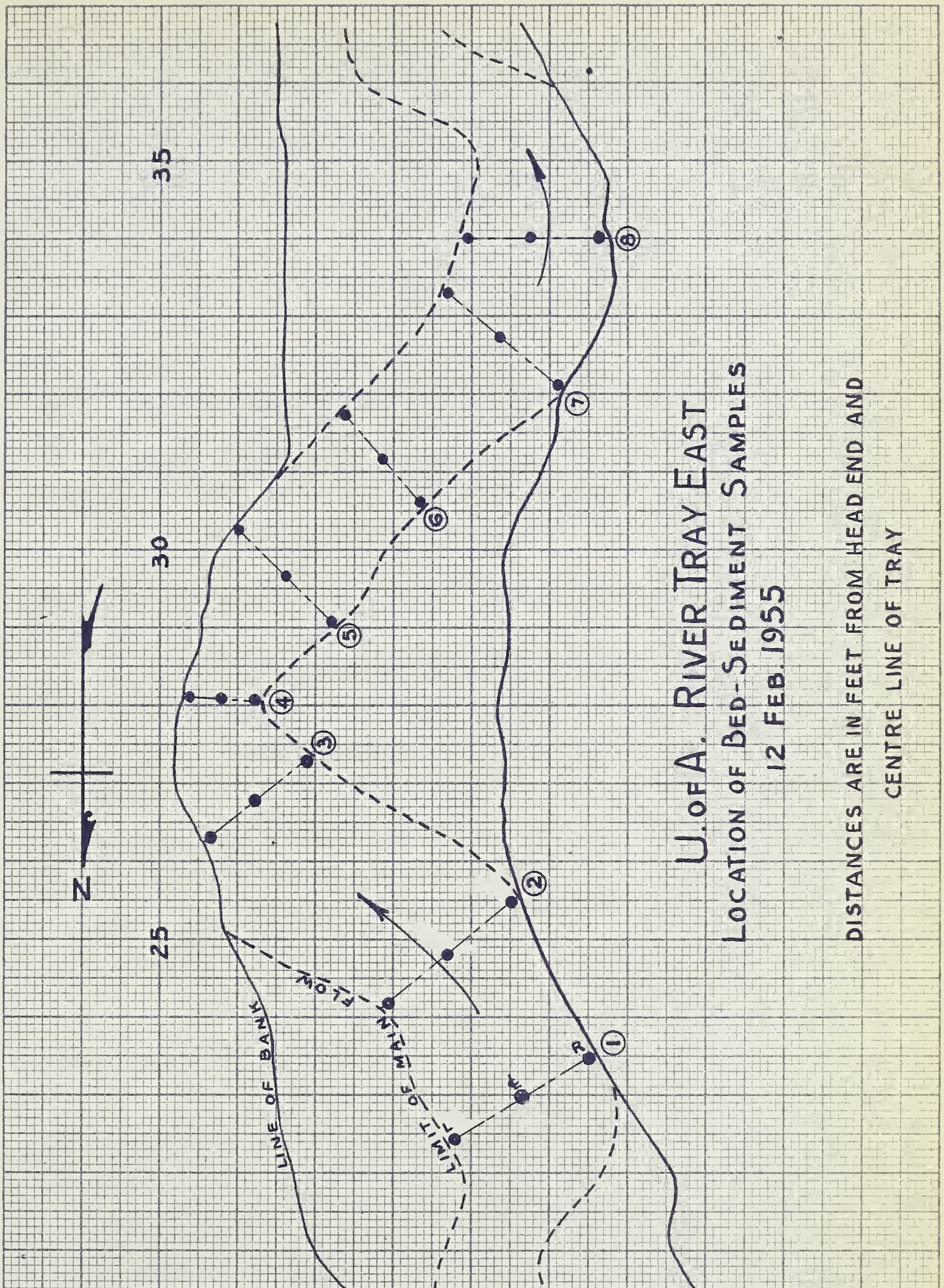
The model spurs were rectangles of plywood placed in a similar location on the model and anchored well into the bank. No attempt was made to reproduce a hydrograph. The changes which took place were due solely to the continuous erosive action of the river under a steady discharge.

The nature of the damage bore a remarkable resemblance to that which occurred on the prototype. While no attempt was made to develop a more successful method of bank protection for this case, the results show clearly that models of this nature can be used to good advantage in the analysis of the qualitative effects of engineering works on rivers.

SAND GRADING AROUND A MEANDER BEND

Plates 11 to 18 give the results of an investigation of sand grading at points around a meander bend. The sampling locations are shown on Plate

The three grain-size curves from each numbered line were plotted together on logarithmic probability paper. It had been hoped that they would illustrate the movement of coarse particles to the inside of a meander bend. No definite trend could be established, due no doubt to the small size of the model.



U. OF A. RIVER TRAY EAST

LOCATION OF BED-SEDIMENT SAMPLES

12 FEB. 1955

DISTANCES ARE IN FEET FROM HEAD END AND CENTRE LINE OF TRAY

PLATE No. 10

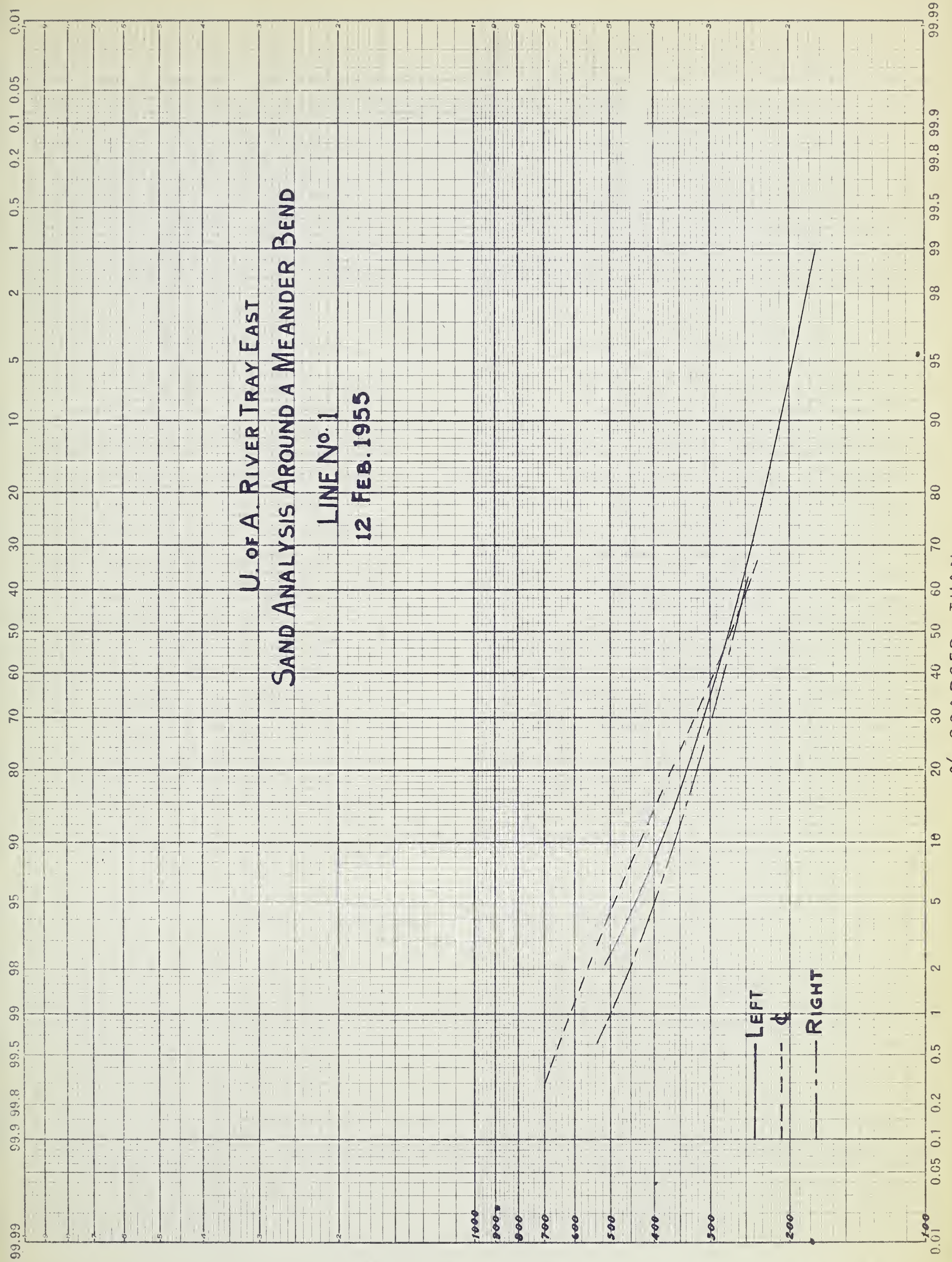
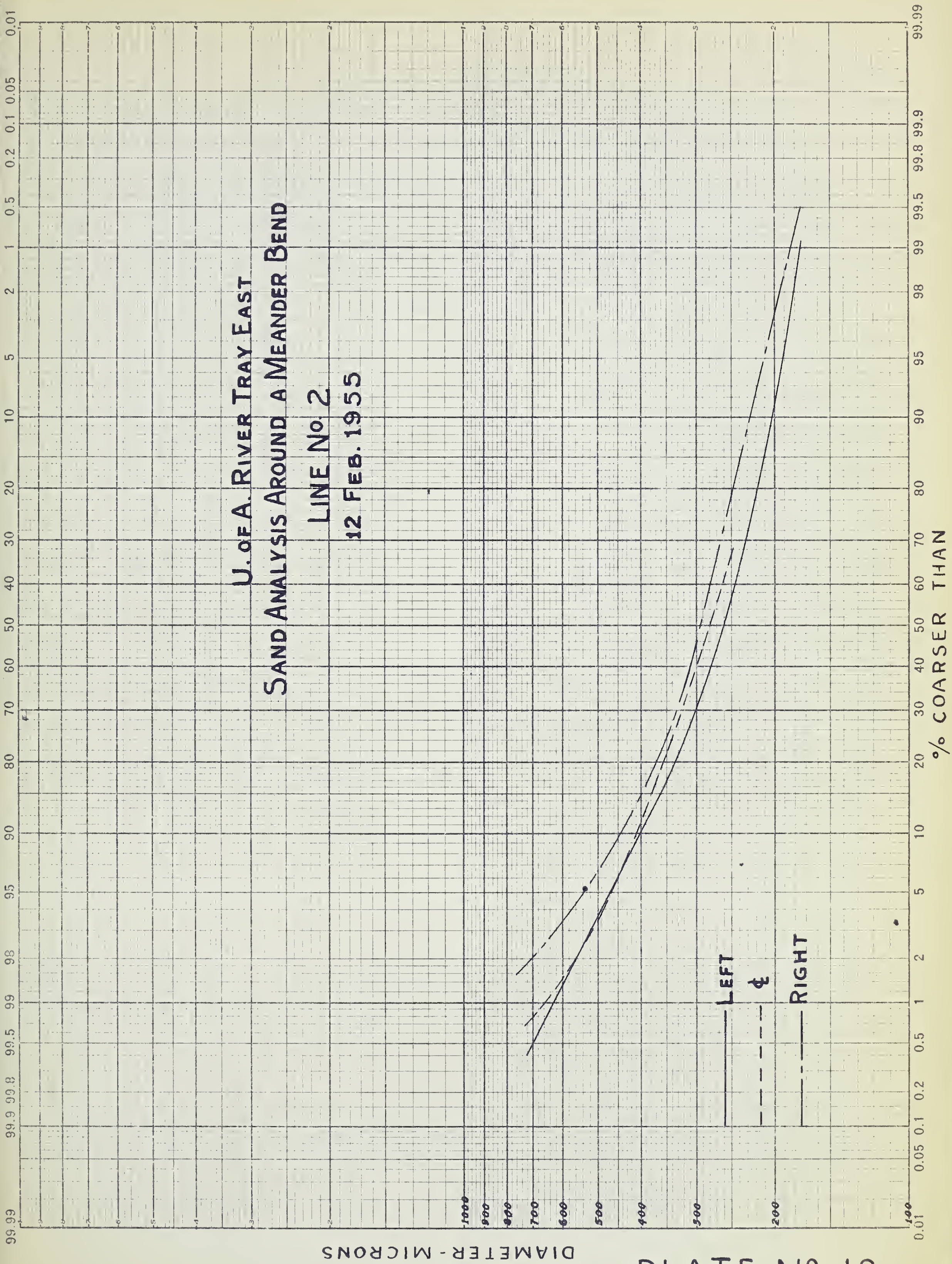

U. of A. River Tray East
Sand Analysis Around a Meander Bend
Line No. 1
12 Feb. 1955
Left
℄
Right
% COARSER THAN
DIAMETER-MICRONS
100
200
300
400
500
600
700
800
900
1000
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
99.99

PLATE No. 11



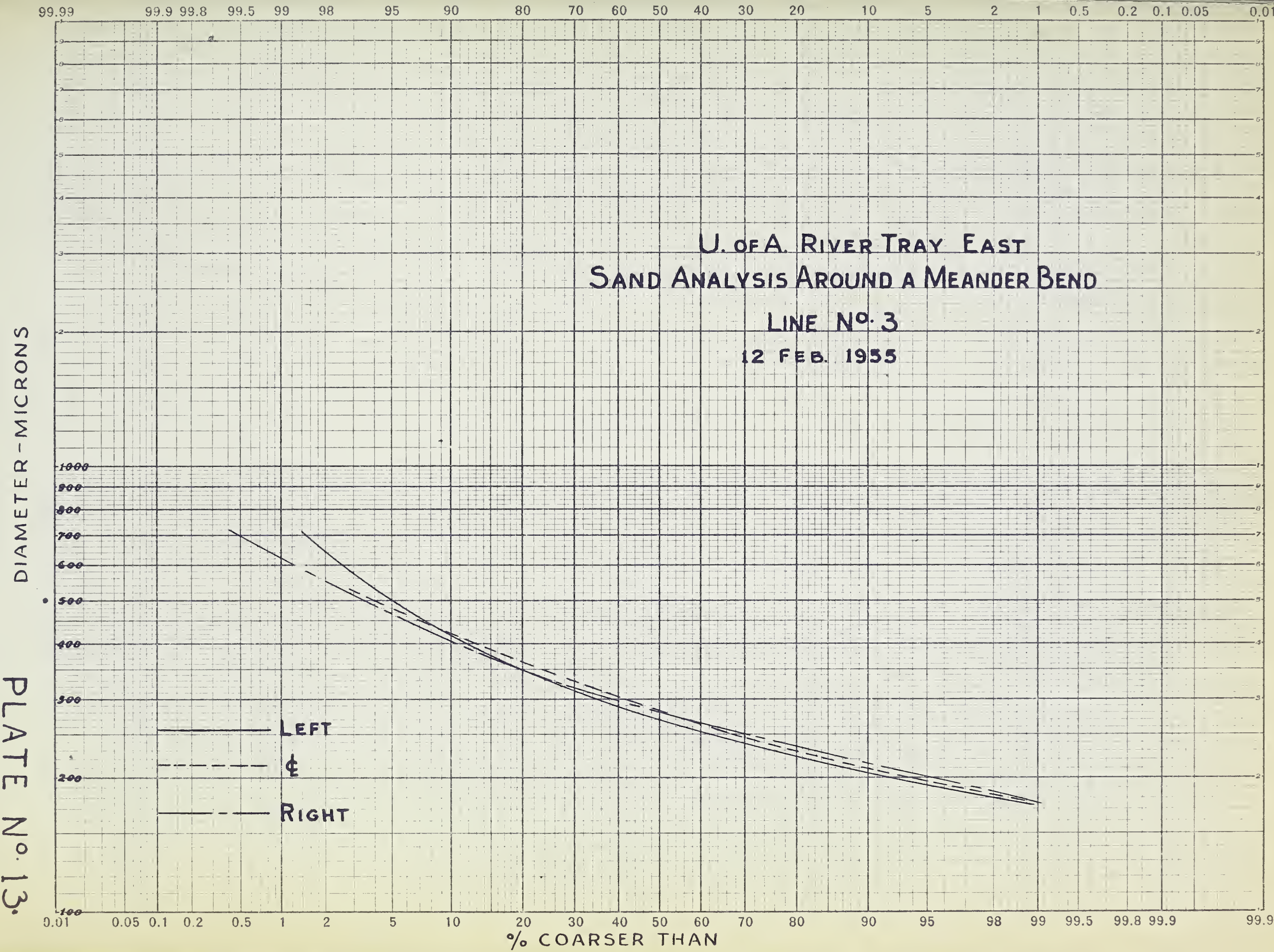
U. of A. River Tray East

Sand Analysis Around a Meander Bend

Line No. 2

12 Feb. 1955

PLATE No. 12



U. of A. River Tray East
Sand Analysis Around a Meander Bend
Line No. 3
12 Feb. 1955

PLATE No. 13.

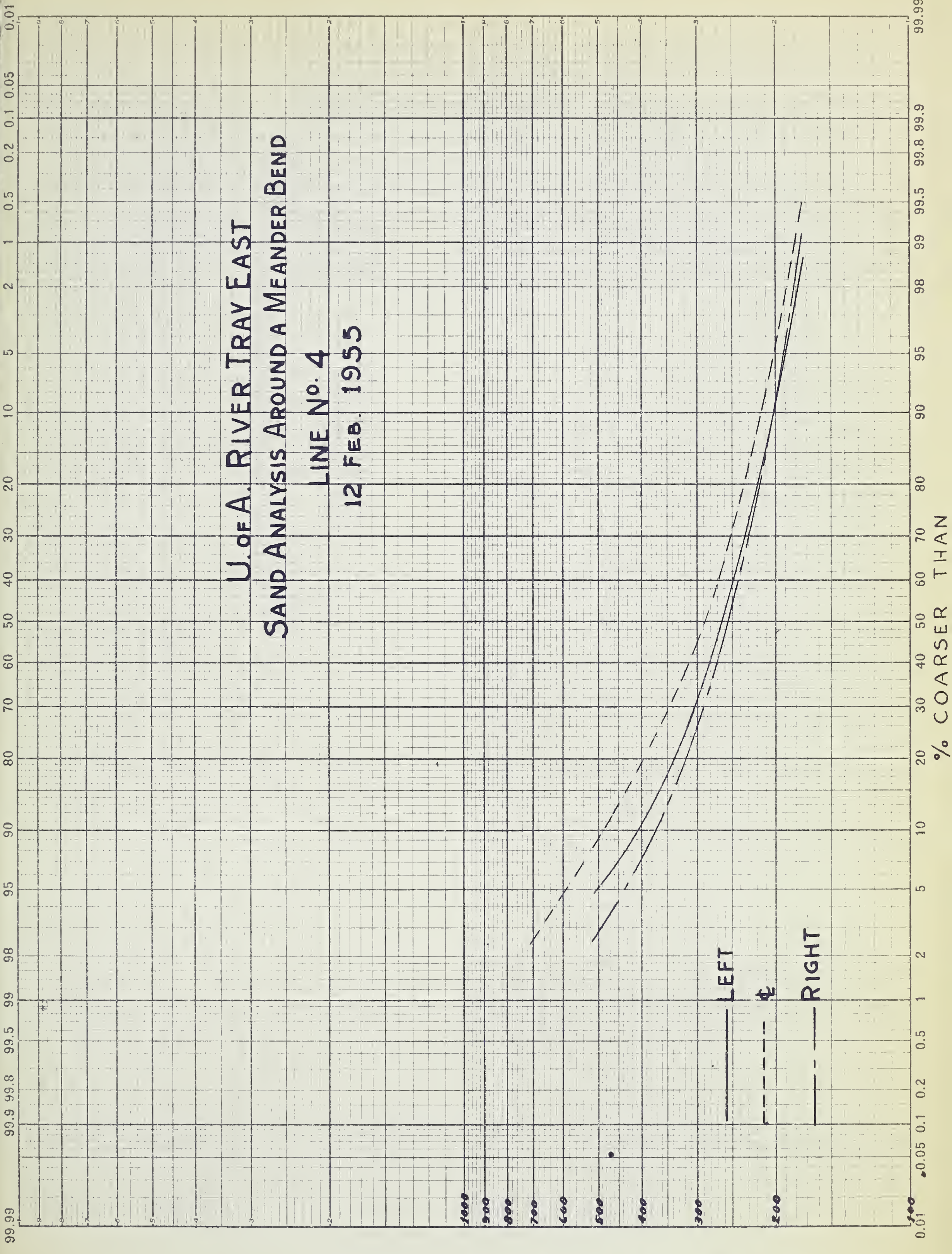

U. of A. River Tray East
Sand Analysis Around a Meander Bend
Line No. 4
12 Feb. 1955
Left
℄
Right
% COARSER THAN
DIAMETER - MICRONS
1000
900
800
700
600
500
400
300
200
100
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
99.99

PLATE No. 14

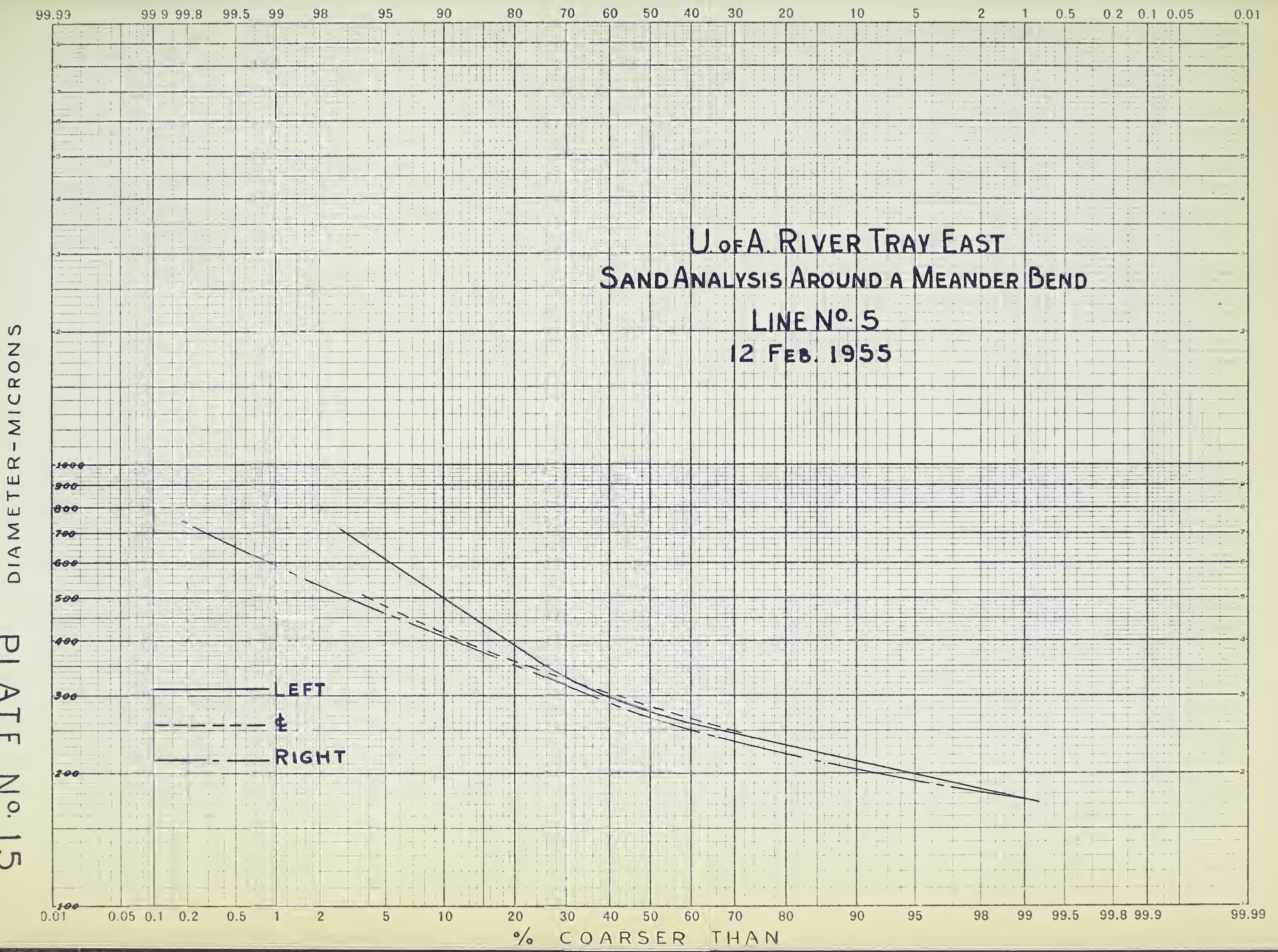

U. of A. River Tray East
Sand Analysis Around a Meander Bend
Line No. 5
12 Feb. 1955
LEFT
℄
RIGHT
DIAMETER-MICRONS
% COARSER THAN
1000
900
800
700
600
500
400
300
200
100
99.99
99.9
99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01

PLATE No. 15

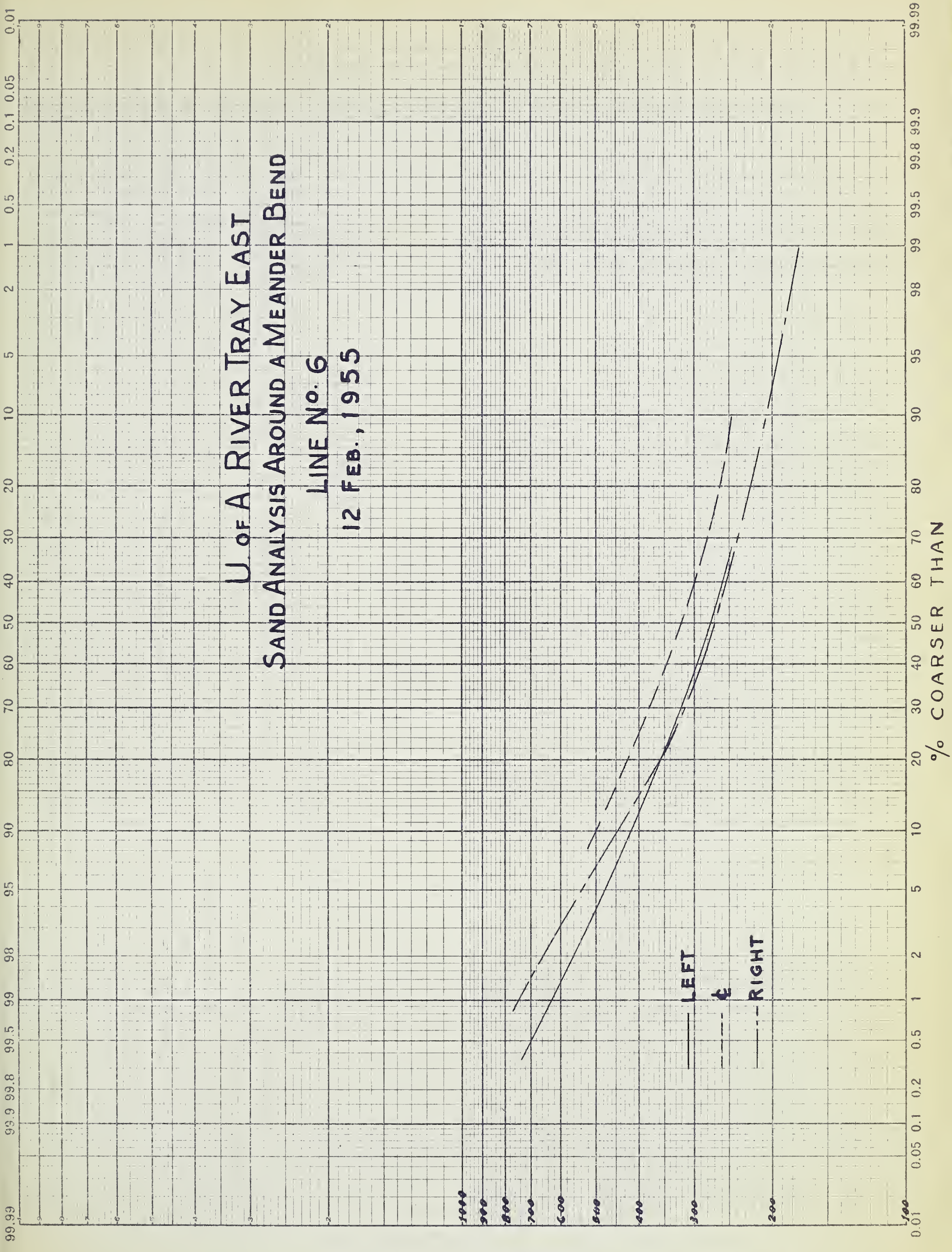

U of A River Tray East
Sand Analysis Around a Meander Bend
Line No. 6
12 Feb., 1955
LEFT
℄
RIGHT
% COARSER THAN
DIAMETER - MICRONS
1000
900
800
700
600
500
400
300
200
100
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
99.99

PLATE No. 16

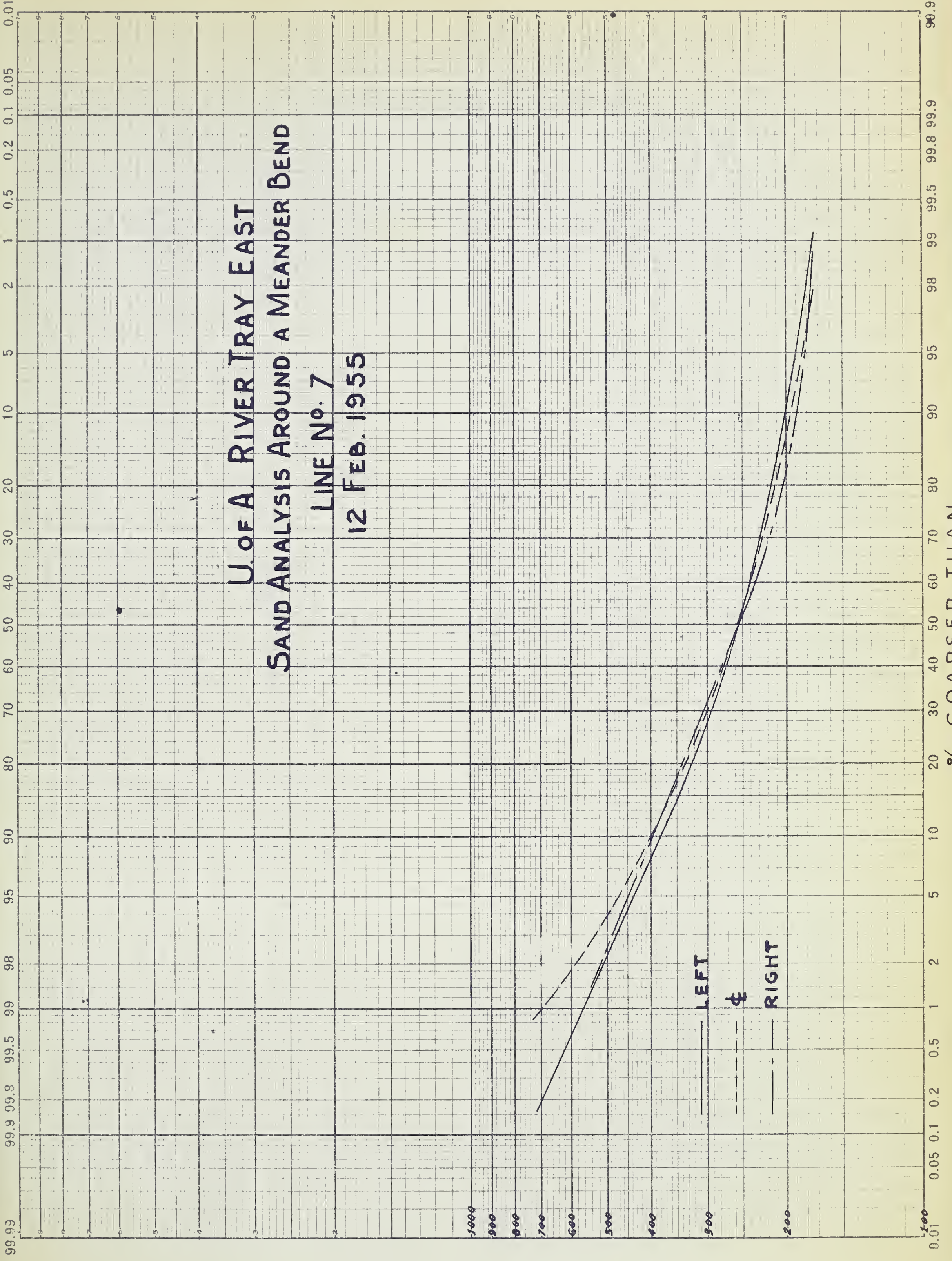

U. OF A. RIVER TRAY EAST
SAND ANALYSIS AROUND A MEANDER BEND
LINE No. 7
12 FEB. 1955
LEFT
℄
RIGHT
% COARSER THAN
DIAMETER-MICRONS
99.99
99.9 99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2 0.1 0.05
0.01
1000
900
800
700
600
500
400
300
200
100

PLATE No. 17

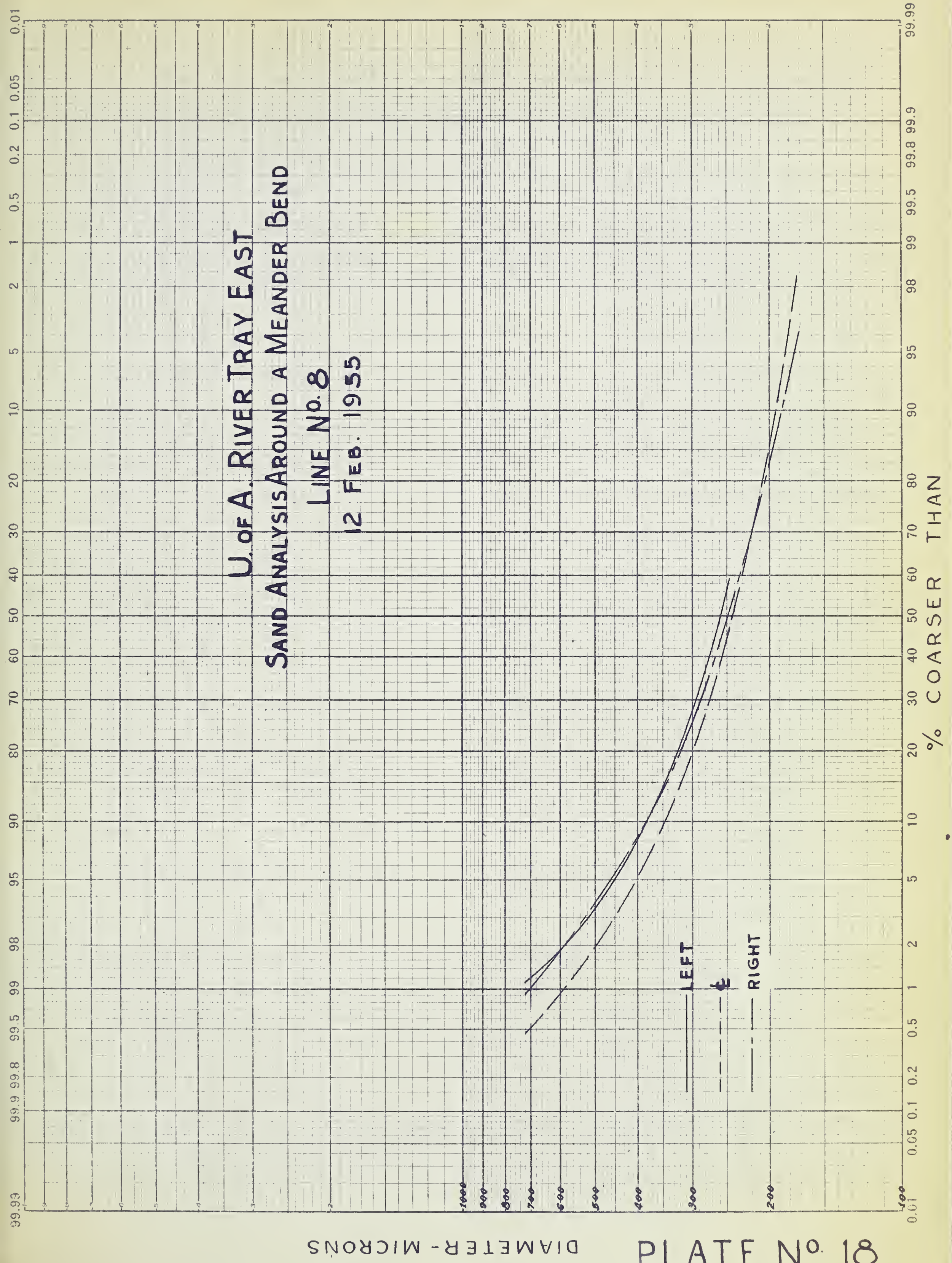

U. of A. River Tray East
Sand Analysis Around a Meander Bend
Line No. 8
12 Feb. 1955
LEFT
℄
RIGHT
% COARSER THAN
DIAMETER - MICRONS
1000
900
800
700
600
500
400
300
200
100
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
99.99

PLATE No. 18

Lines 1, 4 and 6 showed to a fairly marked degree a coarser grading at the centre-line than at the edges. All curves were concave upward, a characteristic claimed for beach sands. (Ref. 6).

CONCLUSIONS

Conclusions related specifically to the experiments are obvious from the foregoing discussion and the illustrative plates. In general it may be said that further evidence was obtained of the utility of the river tray in model studies both for an enrichment of the fund of knowledge of river control and for the study of particular river engineering problems.

Future investigation at the University of Alberta would benefit greatly from a number of equipmental improvements. A device for controlling tailwater levels would make for steadier rates of sediment movement. At present, "slugs" of sand are swept away from the downstream trough and injected into the head reaches whenever an increased discharge is imposed.

A simple measuring weir would be much more satisfactory than the large manometers now used. The latter are prone to clogging while being brought into action, and require careful handling to avoid entrapping air.

Such a weir could be equipped quite simply with a controlling device to keep the discharge constant at any selected value. A more elaborate arrangement to impose a flood cycle at regular intervals would permit a worthwhile expansion of the testing program, particularly in connection with the production of cinematographic records.

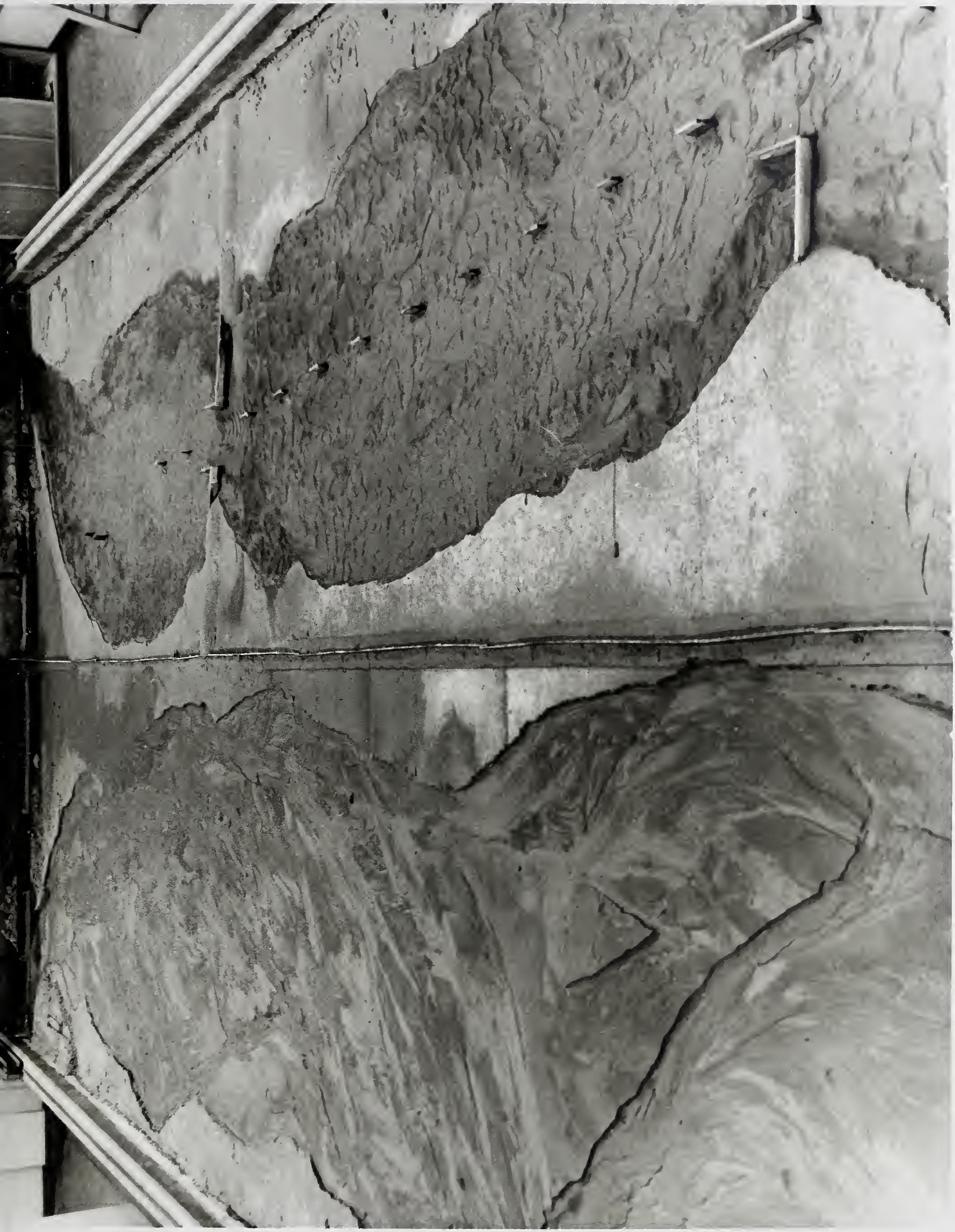


PLATE Nº. 19

PART II - FLUME TESTS

DETERMINATION OF VELOCITY AND DEPTH RELATIONSHIPS FOR FLOW OVER A SAND BED.

PREAMBLE

A great deal of experimental and observational work has been done to determine the laws of sediment transport. Much remains to be done, particularly in reducing the masses of observed data to simple rules usable by the practising engineer.

A review of past flume experiments in the laboratory shows that insufficient thought has been given to the techniques necessary to the production of truly reliable information. In particular, many flume tests have been run for periods which appear to be quite inadequate for the attainment of stability.

No data exist which show how nearly the final adjusted conditions will be reached in a given time. The researcher is therefore hampered in an analysis of existing data by the uncertainty of being able to fix these final values.

One purpose therefore of the present experiments was to develop reliable techniques for this type of flume study, and to determine the order of time required for the production of acceptable data.

It is perhaps surprising at first glance that so little information is available from the numerous experiments which have been done on these lines, and that an analysis of the existing data produces such a wide variation of results. It is intended in this thesis to offer reasons for both these facts.

EXPERIMENTAL OBJECT

One of the parameters of regime canal theory (Ref.7), the bed factor V^2/d, lends itself readily to determination in the laboratory. Because of

the difficulty in measuring bed-load in the field, the laws which govern bed-factor must at present be found from laboratory experiments.

It is known that for canals bearing comparatively small amounts of bed-sediment, the bed-factor may be determined with practical accuracy from the rough rule:

$$F_b = 2\sqrt{D}$$

in which "D" is the median size, in millimeters, of the bed-material.

In addition, an analysis of a considerable number of observations (Ref.8) points to a law of the form:

$$F_b = F_{bo} (1 + a C^n)$$

in which F_{bo} is the bed-factor for the condition of no bed-movement, "C" is the "charge" or ratio of sediment load to water-sediment complex flowing, expressed in thousandths of a per cent by weight, and "a" and "n" are numbers which probably are functions of the sand-grain size.

The flume experiments herein described had as their object the evaluation of F_{bo} for several natural river sands, in order to provide a starting point for future experimental and analytical work in this field.

DETAILS AND ARRANGEMENT OF EQUIPMENT

The flume used has vertical walls of plate glass approximately one foot apart with a total length of eighteen feet. At the upstream end is a stilling basin into which the water discharges. Downstream is a sediment trap (not used in these tests) and a steel-walled compartment from which the flow is returned after passing through the flume. Sediment which has been picked up from the sand bed is recirculated with the water to the upstream end.

Since the tests were concerned with the condition of no bed-movement, the discharge line was extended to a point near the bottom of the stilling basin, in order that sand carried from the downstream end of the flume would not re-enter. A photograph of the arrangements is given on Plate 20.

Two grades of sand were immediately available for test. These were the sands used in the two halves of the river tray, having median grain sizes of approximately 0.27 and 0.47 millimeters. The finer sand was chosen for the first series of tests. A depth of five inches was placed in the flume, levelled and smoothed with a screed, using as a guide a straight line pencilled on the flume wall.

A rotameter was fitted into the discharge line for flow measurement. This meter had been calibrated previously and seemed to offer a simple means of measuring discharges.

In order to minimize the time required for each test, it was necessary to ascertain the probable final depth of flow. To this end the modified regime equation for depth was used:

$$d = \sqrt[3]{q^2/F_b}$$

in which "q" is the discharge per unit of width of flow. Interior flume width being approximately one foot, "q" would be numerically equal to the total discharge per unit of time.

In order to arrive at an accurate mean value of F_{bo} for this sand, it was decided to impose in turn a number of discharges and allow the system to attain equilibrium under the influence of each one. A discharge of 0.08 cubic feet per second was chosen for the first run, with the intention of increasing the flow in increments of 0.02 c.f.s., up to the capacity of the flume pump.

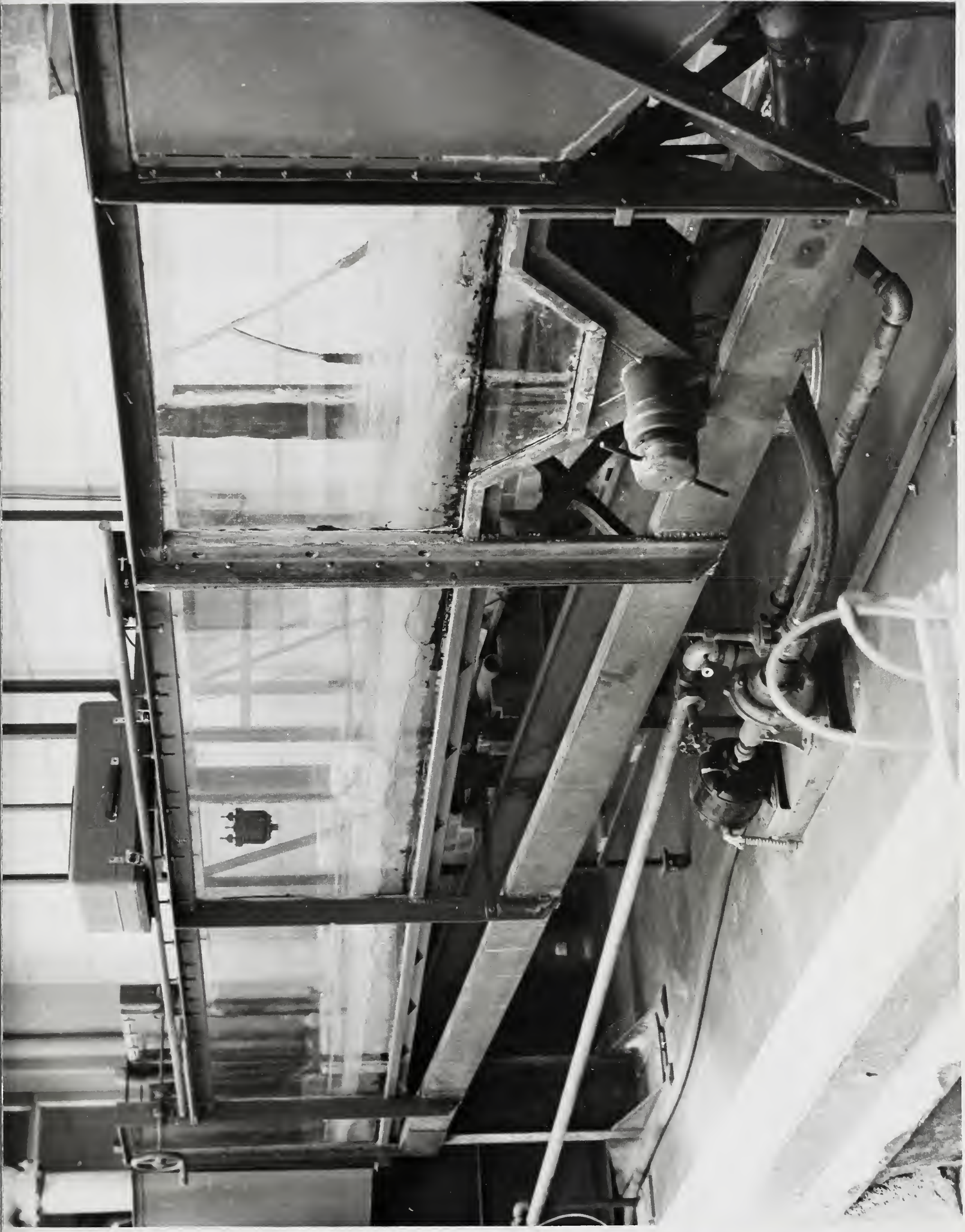


PLATE N°. 20

Preliminary calculations were as follows:

$$F_{bo} \simeq 2\sqrt{D}$$

$$= 2\sqrt{0.27} \simeq 1.4$$

$$d = \sqrt[3]{q^2/F_b} \text{ ft.}$$

$$= \sqrt[3]{\frac{0.08^2}{1.4}} \text{ ft.}$$

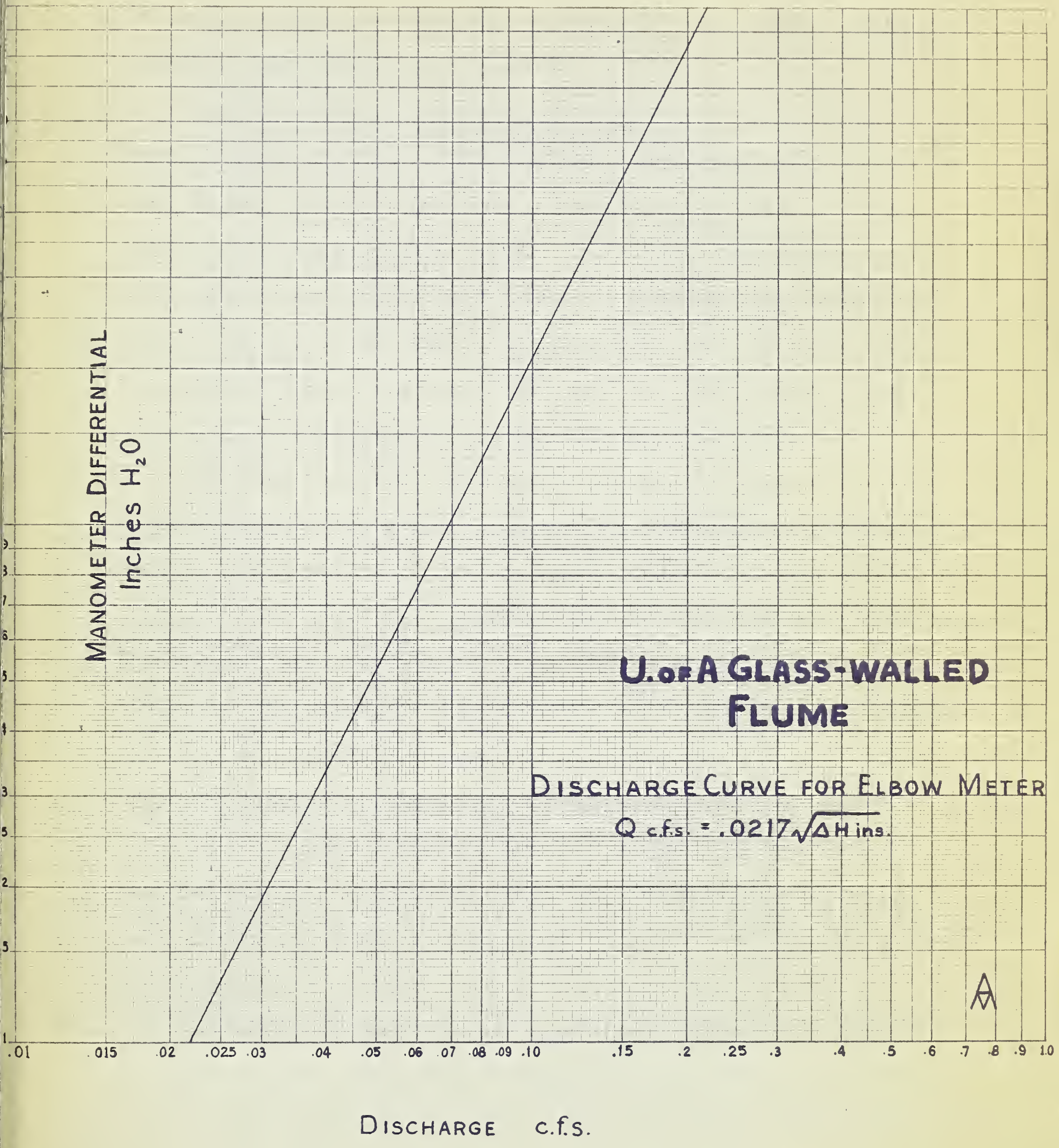
$= 0.166$ ft., or approximately 2 inches

EXPERIMENTAL PROCEDURE

Water was now admitted to the flume to give slightly more than this depth of water over the sand bed, the pump started and the discharge valve set to give 0.08 c.f.s. The water level was then lowered slowly until bed-movement was seen to begin at the upstream end of the flume.

Progress was at first favourable. Dunes formed on the bed and moved downstream in the usual way. After a few hours, however, the sand passing through the rotameter jammed the rotor so that the meter was rendered useless for measuring the discharge. The rotor was cleared a number of times, but it was soon evident that other means would have to be found for discharge measurement. Accordingly, an elbow meter was made and fitted into the discharge line. To this was connected an air-over-water manometer, the rotameter being reinserted temporarily downstream of the elbow for calibration purposes.

A calibration chart for the elbow meter is appended (Plate 21). This device has proven satisfactory from the beginning. The use of 0.4 per cent borax in the water as a rust inhibitor was taken to be adequate insurance against significant changes of characteristics of the elbow meter.



U. of A Glass-walled Flume

Discharge Curve for Elbow Meter

$Q_{c.f.s.} = .0217\sqrt{\Delta H_{ins.}}$

PLATE Nº 21

During the calibration it was found that although the flume pump was rated at 0.2 c.f.s., the maximum discharge obtainable through the system was a little more than 0.15 c.f.s. Thus the tests were limited to a smaller range than had been expected.

The flume was again started, but several weeks were lost due to leaks which developed one by one. The pump had to be stopped each time and the flume drained, allowed to dry and then sealed. In addition, after all major leakage losses had been stopped, tests showed that evaporation and the remaining small losses would have to be replaced by the addition of water. This was accomplished by mounting at the tail end of the flume a reservoir and syphon, the latter equipped with a needle valve, so that water could be added drop by drop at a pre-determined rate.

The first successful trial, using a discharge of 0.08 c.f.s., was begun on February 4th, 1955. It was believed that a period of three or four days would suffice for the attainment of stability. The plan was to outline the bed profile in the test section by pencilling on the glass wall. When the individual waves showed no movement during a period of several hours, the system could be assumed stable. Trial No. 1 took a total of thirteen days to attain stability as gauged by this criterion.

The method of arriving at the final mean depth was as follows:

The water surface line was pencilled on the flume wall and the flow then stopped. The outline of the bed profile was drawn also for both sides of the flume. A trace of these outlines was then made on cross-section paper.

The water surface was lowered carefully by draining and syphoning to expose the bed. Measurements of depth along the centre-line were then taken by sighting through the glass in line with the two water surface marks

on to a scale graduated in inches and tenths. The horizontal distance of each measurement from an arbitrary origin was recorded also.

The areas between water surface and bed along the side were measured by planimeter and mean depths obtained therefrom by dividing by the distance between wave crests near either end of the test section. For the centre-line, the corresponding area was computed from the measured depths and distances, assuming a straight line from crest to trough of each dune. Mean centre-line depth was obtained as for the sides. The average of the three mean depths was taken as the overall mean for the test section.

At the same time, several other characteristic quantities were computed, including average wave length and amplitude, maximum wave height, and the average depth from extremes of high and low.

It was hoped that given sufficient data, a number of useful relationships could be found between the various observed quantities. Thus it might be possible by comparing the average of the extremes of depth on both sides with the "true" mean depth, to arrive at the correction to be applied to this average, and so eliminate the necessity for the tedious work involved in the depth determination. A relationship between discharge intensity, sediment size and wave length and amplitude might also be extracted from the observations.

Since the time required for full self-adjustment was two weeks for the fine sand, it was obviously impossible to obtain sufficient data to draw any usable conclusions regarding the above interrelationships. Future work should reveal their existence and so provide a valuable addition to what is known in this field.

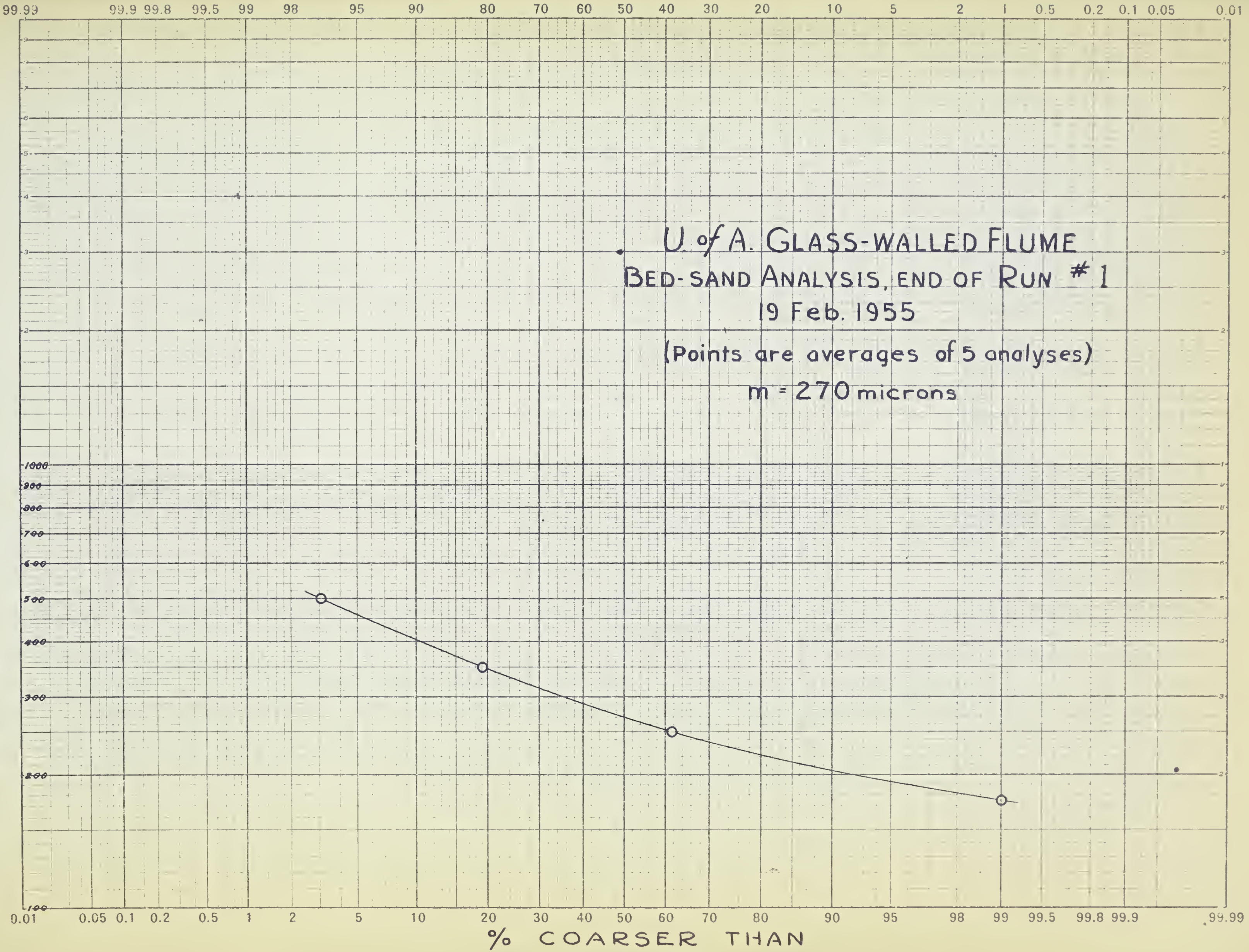
TREATMENT OF DATA

A careful measurement of the flume test section showed its average width to be 12.4 inches or 1.033 feet. Unit discharges were obtained therefore by dividing total discharge by this figure. Velocity was the unit discharge divided by mean depth. The bed-factor V^2/d was then computed.

Average depths were calculated also from the measurements to the highest and lowest points on the two flume sides, and the values shown in the third column of Table III, Page 18. Column 4 shows the averages of the depths to all the troughs and crests along both sides and the centre-line. In Table IV are given the average wave length and amplitude and maximum wave amplitude for each run.

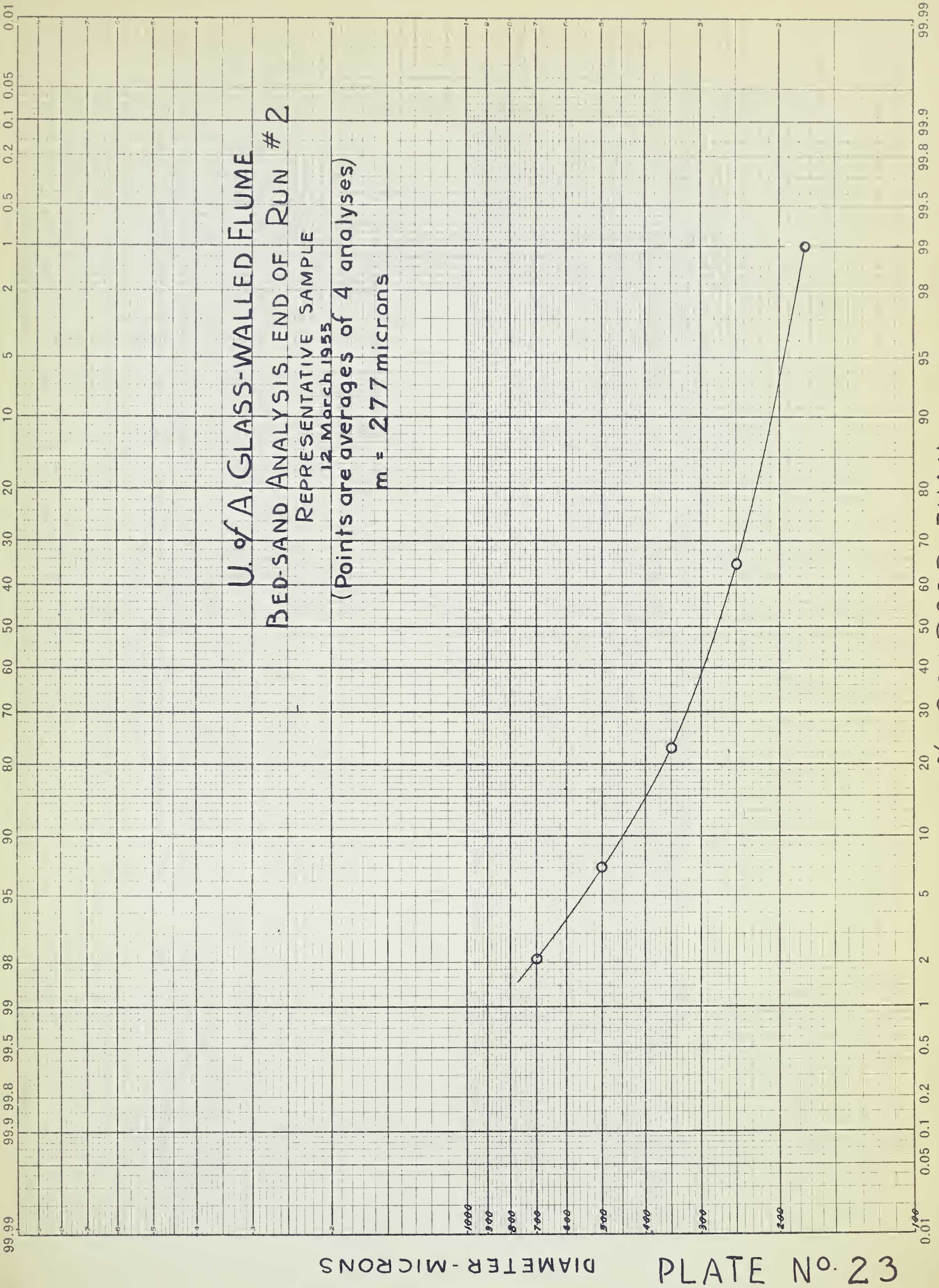
In order to check the rough rule $F_b = 2\sqrt{D}$, samples of the bed-material were taken at the end of each run and analyzed, and the value of $\frac{F_b}{\sqrt{D}}$ computed. After Run No. 2, samples were taken also from the surface of a crest and a trough in order to show the variations of median size. All samples were analyzed in the visual accumulator analysis recorder (Ref. 4). Grain size curves on logarithmic probability paper are given on Plates 22 to 26.

The experimental results are tabulated on Page 18.



U of A. GLASS-WALLED FLUME
BED-SAND ANALYSIS, END OF RUN # 1
19 Feb. 1955
(Points are averages of 5 analyses)
m = 270 microns

PLATE No. 22



U. of A. GLASS-WALLED FLUME

BED-SAND ANALYSIS, END OF RUN #2

REPRESENTATIVE SAMPLE

12 March 1955

(Points are averages of 4 analyses)

m = 277 microns

PLATE No. 23

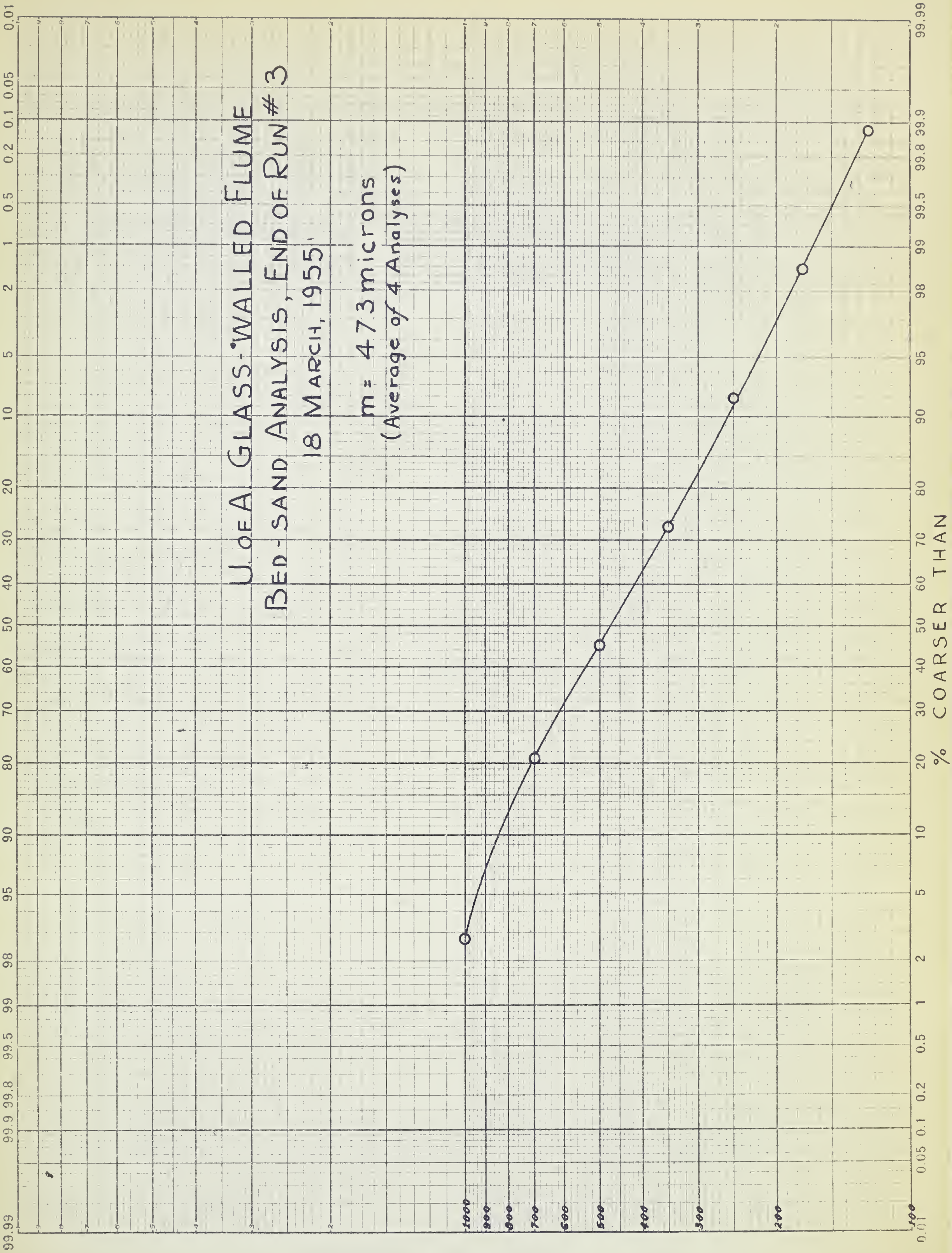

U. of A Glass-Walled Flume
Bed-sand Analysis, End of Run # 3
18 March, 1955
m = 473 microns
(Average of 4 Analyses)
% COARSER THAN
DIAMETER - MICRONS
1000
900
800
700
600
500
400
300
200
100
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
99.99

PLATE No. 24

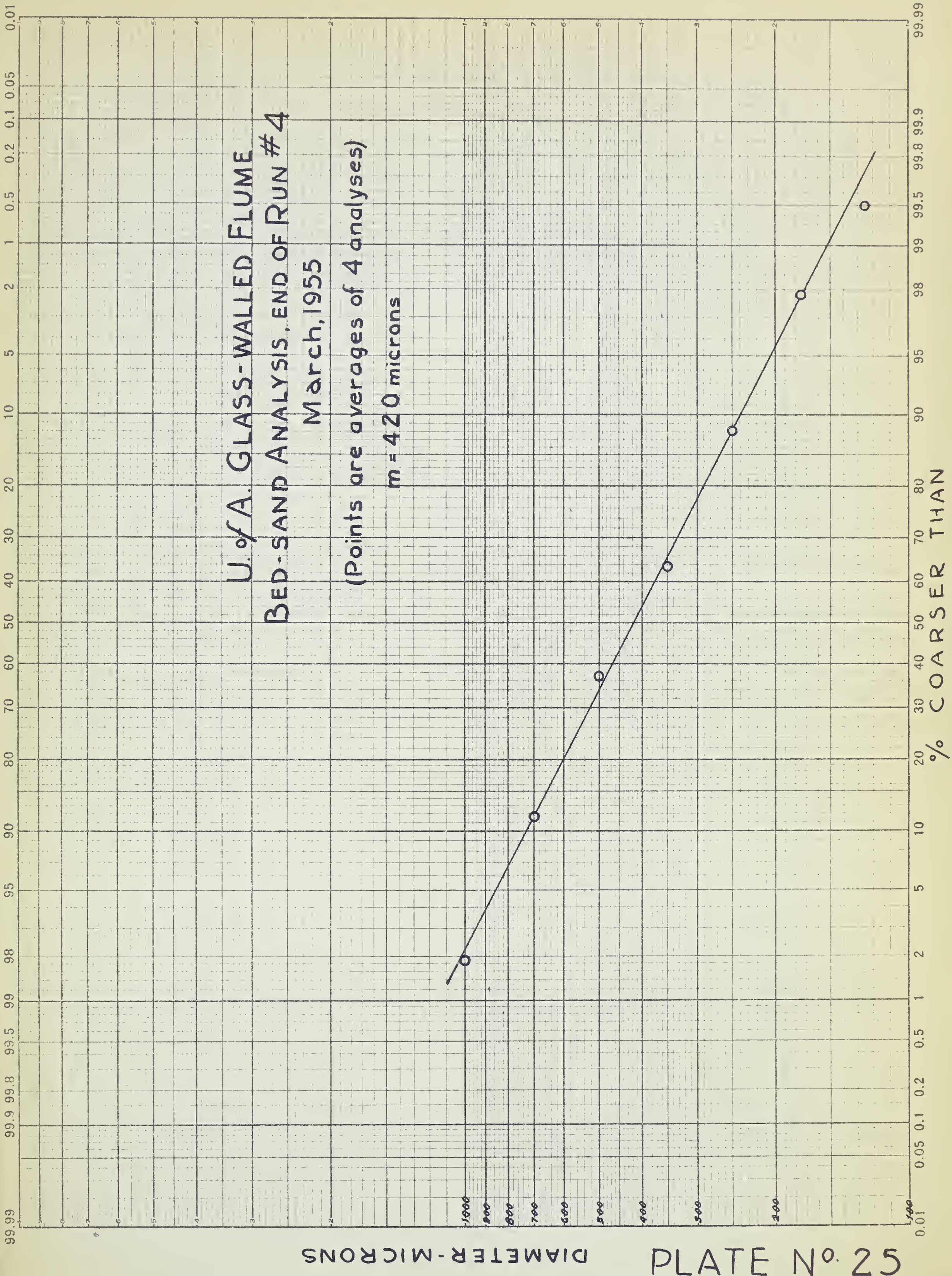
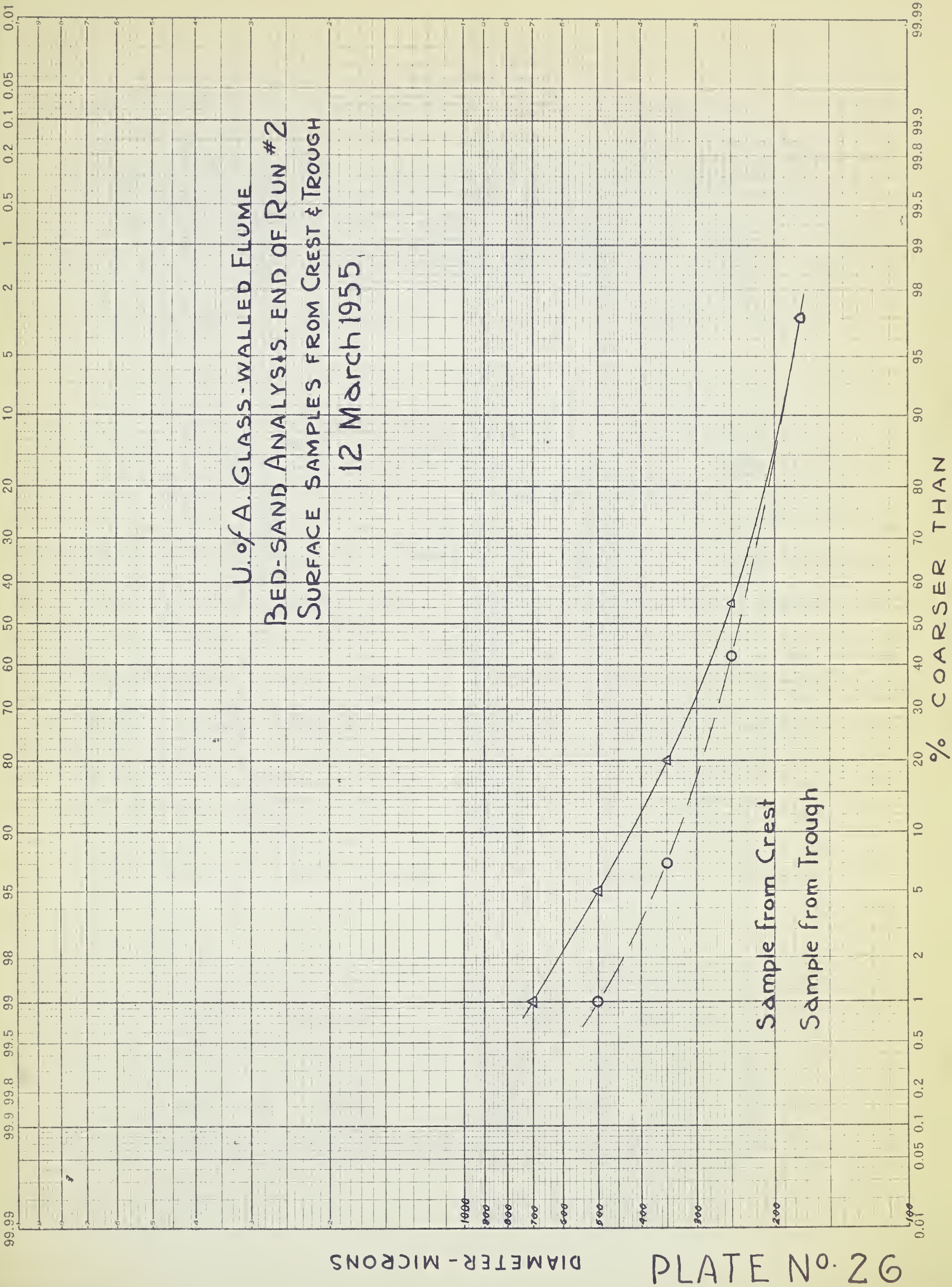

U. of A. GLASS-WALLED FLUME
BED-SAND ANALYSIS, END OF RUN #4
March, 1955
(Points are averages of 4 analyses)
m = 420 microns
% COARSER THAN
DIAMETER-MICRONS
1000
900
800
700
600
500
400
300
200
100
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
99.8
99.9
99.99

PLATE No. 25


U. of A. GLASS-WALLED FLUME

BED-SAND ANALYSIS, END OF RUN #2

SURFACE SAMPLES FROM CREST & TROUGH

12 March 1955

PLATE No. 26

TABLE I

Observed Data

Run No.	Discharge "Q" c.f.s.	Discharge "q" c.f.s./ft.	Mean Depth "d" ft.	Velocity "V" f.p.s.
1	0.08	0.077	0.180	0.429
2	0.10	0.097	0.208	0.466
3	0.10	0.097	0.182	0.533
4	0.15	0.145	0.219	0.662

TABLE II

Run No.	Median Grain Size "D" mm.	Bed-Factor $F_b = V^2/d$	$2\sqrt{D}$	$\frac{F_b}{\sqrt{D}}$
1	0.270	1.02	1.04	1.97
2	0.277	1.05	1.05	1.99
3	0.473	1.56	1.38	2.26
4	0.420	2.00	1.30	3.09

TABLE III

Run No.	Mean Depth by Areas, "d" ft.	Ave. of N. & S. Extremes, ft.	Overall Average of Measured Depths, ft
1	0.180	0.178	0.180
2	0.208	0.203	0.208
3	0.182	0.184	0.183
4	0.219	0.223	0.220

TABLE IV

Run No.	Ave. Wave Length, ft.	Ave. Amplitude, ft.	Max Amplitude, ft.
1	0.458	0.050	0.121
2	0.633	0.081	0.126
3	0.665	0.052	0.105
4	0.607	0.054	0.138

CONCLUSIONS

The results show first of all that experiments of this nature cannot be expected to give reliable results in a matter of a few hours. This has two important implications. One is that many previous flume tests have been run for totally inadequate periods of time and that the data resulting therefrom are at best unreliable. The other is that in any extensive program of experimental work in this field, no great masses of data can be expected in one short period. The experimenter must be prepared to set up his tests with care and then to wait patiently for results.

It is obvious too that in flume tests such as those described, complete steadiness of operation is required since the time involved is considerable. Any loss of water, even by evaporation, must be allowed for if the system is to reach a true equilibrium. Supply and control devices must be capable of fine adjustment and steady operation for long periods of time.

The bed-factors obtained, as given in Table 2, do not bear the expected relationship to the corresponding median grain sizes. While the small number of values recorded obviates the drawing of any definite conclusions, it is possible that the discrepancies are due to the shortness of the flume. That is, the flume was perhaps not long enough to provide the required entry length upstream of the test section. This entry length would then appear to be a function of flow velocity and grain size.

The results of other flume experiments, as for example, those described in Ref.9, may be in error for the same reason. It is hoped that the long flume, now under construction at the University of Alberta, will supply the answers to these and many other questions. The results reported here may then be confirmed or disproved.

It is possible also that suspended charge, seen to be greater for the coarse sand, had an appreciable effect on the bed-factor. This suggests the taking of suspended sediment samples in future.

An examination of Table III shows that averaging the measured depths along the flume sides and centre-line gives a mean depth almost identical with that obtained from the area measurements. The mean of the two extremes at the sides also comes reasonably close to the true mean depth. If these results are confirmed by future experiments, it will be feasible to obtain by one of these simple methods the mean water depth without the tedious work of area measurement.

The table of wave-lengths and amplitudes is included as a starting-point for a file of useful information of this nature, whose utility will be evident.

For future tests in the University of Alberta glass-walled flume it is suggested that the whole system be checked for leakage losses with no sand on the bed. This will simplify the repairs necessary and save a great deal of time. Calibration of the elbow meter or other measuring device should also be done at this time. A check on the present arrangements will be necessary because of probable rust or scale accumulation before the next use is made of this apparatus.

APPENDIX

Summary of Regime Formulae

The following terms are used:

D - Depth of flow from water surface to bed in ft.

W - Mean width in terms of depth D, so that WD = cross-sectional area of flow; in ft.

S - Slope of channel; nondimensional

Q - Discharge in c.f.s.

ν - Kinematic viscosity in sq. ft. per sec.

V - Mean velocity of flow in ft. per sec.

T - Time in seconds

r - Scale ratio

q - Discharge intensity, $\frac{Q}{W}$ in c.f.s./ft.

The bed factor is defined by

$$F_b = \frac{V^2}{D}$$

The side factor is defined by

$$F_s = \frac{V^3}{W}$$

$$W = \sqrt{\frac{F_b Q}{F_s}}$$

$$D = \sqrt[3]{F_s Q/F_b{}^2} \quad = \sqrt[3]{q^2/F_b}$$

$$S = \frac{F_b{}^{5/6} F_s{}^{1/12}}{2080 Q^{1/6} c}$$

where c is $(10^{-5}/\nu)^{1/4}$

$$(T)_r \overset{\frown}{=} \left(\sqrt[3]{\frac{F_b Q}{F_s{}^2}}\right)_r$$

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